HYDRAULIC MANUAL.

PART I.

CONSISTING OF

WORKING TABLES

AND

EXPLANATORY TEXT,

INTENDED AS A

GUIDE IN HYDRAULIC CALCULATIONS

AND

FIELD OPERATIONS.

BY

LOWIS D'A. JACKSON, A.I.C.E.

AUTHOR OF "A CURVE-BOOK."

LONDON:
W. H. ALLEN & CO., 13, WATERLOO PLACE, S.W.
1875.

PREFACE TO THE THIRD EDITION.

In presenting this third edition to the public, it is unfortunately my duty to apologise to those interested in the work for the delay that has taken place in its publication; this, however, has been due to circumstances over which I discovered eventually that I had but little control. To avoid disappointing the public generally, and prevent them from expecting to find anything in this book that is not in it, it is necessary to state the intentions and scope of the work.

The object of this Manual is to aid the hydraulic engineer in his calculations by means of a collection of working tables based on the most improved modern principles, and by a small amount of text setting forth these principles and giving all the necessary formulæ in a concise manner; also to serve as a guide in hydraulic field operations by giving short résumés of the modes adopted in the field by the engineers whose experiments have been particularly eminent in producing practical and theoretical results.

A few miscellaneous paragraphs on various hydraulic subjects are also attached with the hope that some of them may prove of interest, and that others may show the state to which the collective experience of the past has arrived, unsatisfactory though it may be in many instances.

In such a work, which is necessarily a compilation, the principal object has been to avoid as much as possible any attempt at originality, which might defeat the object of the Manual, and at the same time to incorporate the most recent information in the form most convenient for practical application, while not neglecting any of the more ancient but still useful modes and formulæ of calculation that have not yet been superseded.

The works principally consulted, and from which extracts and information have been taken, are—D'Aubuisson's "Hydraulics," D'Arcy and Bazin's "Recherches Hydrauliques;" the "Cultur-Ingénieur," for 1869 and 1870, containing the valuable articles of W. R. Kutter, of Bern; Claudel's Tábles, constructed on the system of Dupuit; the Mississippi Report of Captains Humphreys and Abbot; the Lowell experiments by Francis; the "Hydraulics of Great Rivers," by J. T. Révy; also, in a small degree, Box's "Hydraulics," Neville's well-known work on the same subject, Stoddard's and Dwyer's works, Spon's "Dictionary of Engineering," Hurst's Manual, some ancient numbers of various periodicals and cyclopædias, and some articles in the Roorkee professional papers, by Colonel Dickens, Mr. Burge, and Mr. J. H. E. Hart.

In addition, my thanks are especially due to the latter gentleman, for placing at my disposal his valuable MSS. on dams and walls, and to a friend for his on towage.

The Second Part of the Manual, annexed to the first in accordance with the wishes of the Secretary of State for India, consists entirely of hydraulic and meteorological statistics, the former principally, and the latter altogether, Indian.

The hydraulic statistics may be useful for reference in connection with works of irrigation, storage, and river-improvement in any part of the world, but more especially in hot climates. It has not, however, been found advisable to incorporate with them any statistics of irrigation suitable to England or to cold climates generally, because, though the irrigationists in England have certainly achieved an important success in demonstrating unmistakeably that theirs is the only practical mode of dealing with sewage, and are likely to carry out such matters on a more extended scale; yet, in the first place, this system of sewage irrigation differs greatly from the more simple watering practised in warm countries; and, in the second place, the experiments and results obtained at Croydon, Barking, Merthyr Tydvil, Aldershot, and the few other places, do not appear to admit of satisfactory comparison, or to afford a guidance useful under other local circumstances and conditions, either as regards amount or intermittency of supply.

Such of the hydraulic statistics as relate to India were mostly collected by myself personally, in the various provinces of India from the different local officials and Government records, and reduced to their present shape. These cannot be expected to be of so much interest to engineers of exclusively home practice as to those of more extended experience; and again, the results shown by them may appear, in the eyes

of many, to be small in comparison with what might have been done in India under a more favourable administration. While the latter is doubtless true, its counterpart is no less so; it is also surprising that so much has been done under such extreme administrative and financial difficulties; in fact, there is every reason to believe that, had it not been for the energy and great administrative abilities of the former Inspector General of Irrigation, General Strachey, all irrigation works in India would probably have remained at a standstill from 1869 till now, and perhaps longer.

At present, the older canals are being rectified, and new works gradually carried out. The results are not entirely satisfactory in all cases, nor is it possible that they should be; they are, however, on the whole, extensive results, showing an actual and a progressive development of irrigation not existing in any other country, which have not hitherto been collected and impartially set forth in a form conveniently for reference. In the present edition, some modern additions, relating to the years from 1870 to 1873, have been made from India Office records, kindly placed at the disposal of the author by the Under Secretary of State for India. Such statistics as relate to England, France, Italy, and Spain have, in every case, the source from which they were taken mentioned with them.

In all of them, whether tabular or in the form of brief accounts, the object has always been expressly to avoid introducing anything simply because it might be of interest, and to limit myself to simple facts and achieved results that may be useful to engineers for reference. In one or two cases rather doubtful statistics have been introduced, to which foot-notes are attached: this, however, was unavoidable under the circumstances of the case, which were particularly difficult, the voluminous records of India, both at home and abroad, being generally destitute of anything approaching to a catalogue raisonnée, although filed and indexed, according to certain principles, with extreme care. The difficulties, then, had to be overcome in the first place by wading through an immense quantity of matter in order to obtain but a few facts, and in the second place by availing myself of the kind aid of several officials, which materially shortened that labour: to these, therefore, and more especially to the present Secretary in the Geographical Department of the India Office, and to Dr. Macnamara, of Calcutta, I beg to offer my best thanks.

The Indian meteorological statistics here given were also, with the exception of those from 1871 to 1873, collected in India by myself, being supplied by the various meteorological reporters, and after-

wards reduced and worked into the present form as most suitable for reference for engineering purposes. They are the first general collection yet made, and include rainfall statistics of all India, and other meteorological statistics of use to the engineer. For the principal portion of them I am indebted to Mr. Blanford of Calcutta, Mr. Chambers of Bombay, and Dr. Murray Thompson of the Panjab: those for the Madras Presidency, excepting the older rainfall data, are unfortunately less complete. The remarks on the meteorology of India, drawn up by myself, are offered as a general account and explanation of the meteorological conditions of India as far as they are at present known.

With regard to the alterations effected in this edition, they principally consist of replacing two or three of the former working tables by new ones, and adding such new tables as the modern system of Kutter absolutely requires: the appendix of miscellaneous tables and data, which are taken from various works and other sources, is slightly enlarged; the text is generally re-written or re-arranged, some additions being made to the article on modules, including a description of a new module of the author's. The hydraulic statistics, as well as the Indian meteorological statistics, have been increased by all such matter as has reference to data available only since the author's departure from India in 1872; the sole matter expunged being the description of the author's evaporameter.

L. D'A. J.

ROYAL INSTITUTION, ALBEMARLE STREET,
1st March, 1875.

PART I.

TEXT.

Chapter I.—Explanation of the Principles and Formulæ adopted in Calculation and applied in the Working Tables.

Chapter II.—On Field Operations and Gauging; with brief Accounts of the Methods adopted by various Hydraulicians.

Chapter III.—Paragraphs on various Hydraulic Subjects.

WORKING TABLES.

MISCELLANEOUS TABLES.

8

1

PART I.

CHAPTER I.

EXPLANATION OF THE PRINCIPLES AND FORMULÆ ADOPTED IN CALCULATION AND APPLIED IN THE WORKING TABLES.

---

1. Hydrodynamic Theories. 2. Notation and Symbols. 3. Rainfall, Supply, and Flood Discharge. 4. Storage. 5. Discharges of Open Channels and Pipes. 6. Section of Channels and Pipes. 7. Other Theories of Flow. 8. Velocities in Section. 9. Bends and Obstructions. 10. Discharges of Sluices and Weirs. 11. Discharge from Basins, Locks, and Reservoirs. 12. Application of the Working Tables.

---

1. HYDRODYNAMIC THEORIES.

THE science of hydraulics, yet in its infancy, may be said to depend, as far as its practical application by the hydraulic engineer is concerned, on a combination of certain known laws with the empirical results of observation and experiment; the former few in number, and eliminated principally by the philosophers and mathematicians of the past; the latter also few, and, if we except the old observations which were carried out on a very petty and limited scale, exceedingly modern. Previous to the experiments of d'Arcy in 1856, little was known about the velocities and discharges through pipes; until the operations of Captains Humphreys and Abbot on the Mississippi in 1858, the discharge of large rivers was a comparatively unex-

plored subject; in 1865 the experiments of Bazin led the way to a more accurate knowledge of the discharges and velocities of open channels. Before this time the less important subjects alone had been investigated to any practical purpose, such as the vena contracta, the discharges through small orifices, over certain forms of overfall, and through short and small pipes, the discharges from reservoirs, and the velocities in troughs 18 inches wide. There was, however, plenty of theory, and a large number of formulæ, some of them exceedingly complicated in form, mostly resulting from a number of superimposed theories, the more ancient of which were based on very limited experiments: in fact, the mode often adopted seems to have been to assume a new form of formula, and to prove it by a few partial experiments, a principle worthy of ancient soothsayers, and which, had it been further supported by traditionary and name-reverencing hydraulic schools of believers, could only have resulted in prolonged and permanent error. At present even, a reference to some works comparatively recently published in England will show formulæ to be supported by a most heterogeneous collection of experimental data; discharges of pipes irrespective of their material or internal surface, of large and small rivers irrespective of the quality of their beds and the bends in their courses, of canals in any material, down to wooden troughs, all seem to prove the correctness of a fixed formula having an unvarying constant coefficient: other works again having greater accuracy of result in view go to the opposite extreme in method, and recommend the adoption of two distinct formulæ for cases in which the principle involved does not even seem to vary in the least, as for instance, in discharges through pipes with low velocities, a formula distinct from that for those with high velocities is often adopted; this, amounting to a method of successive

approximation imperfectly worked out, is almost as unfortunate as the other. From a continuance of this, however, the modern experiments have already saved us to a great extent, and further and more extended experiment will probably relieve us from it altogether.

At present, therefore, the hydraulic engineer is more dependent for correctness of calculated result on the so-called empirical data obtained by experiment, and put into convenient form, than on any purely mathematical theories or laws. The correct application of all known mechanical laws cannot, however, fail to be valuable in cases admitting of them; those relating purely to hydrodynamics are comparatively few, and the most important and best known of them are the three following:—

First. If fluid run through any tube of variable section kept constantly full, the velocities at the different sections will be inversely as the areas, or

$$A\,V = A'V'.$$

This theory of uniformity of motion is in practice supposed to hold good with reference to mean velocities of discharge; which is actually little more than assuming a theoretical velocity that will fulfil the conditions of the law, in order to render calculation convenient. There is no reason to believe that actual velocities in a tube of variable section would all vary inversely with the area of cross section; hence this theory is not one that throws any light on the laws of absolute velocity.

Second. The velocity of a fluid issuing from an orifice in the bottom of a vessel kept constantly full, is equal to that which a heavy body would acquire in falling through a space equal to the depth of the orifice below the surface of the fluid, which is called the head on the orifice; or by way of formula

$$V = (2g\,H)^{\frac{1}{2}}$$

where H = the head, and g = force of gravity. The quantity g represents the accelerating force of gravity, which varies at different places on the earth's surface and elevations above the mean sea level, and is also affected by the spherical eccentricity of the earth at the place, a quantity that again varies with the latitude; above the earth's surface g varies inversely with the square of the distance from the earth's centre, below the earth's surface direct with the distance from the earth's centre; to obtain the exact value of g, d'Aubuisson's formulæ applied to English feet are—

$$r = 20\,887\,540\ (1 + \cdot 001\,64 \cos 2\,l)$$

$$g = 32{\cdot}1695\ (1 + \cdot 002\,84 \cos 2\,l)\left(1 - \frac{2e}{r}\right).$$

The values of this formula for different latitudes and elevations are given in Working Table No. I., and the values of g, obtained from observation at different latitudes, are given in Table No. I. of the Hydraulic Statistics. For purposes of ordinary calculation in England, and hence throughout these tables, g is generally taken as 32·2 feet per second; in India, however, it would be more correct to use 32·1; but the convenience of using English data will probably outweigh that of this exactness until the science of hydraulics can produce far more accurate results than now.

The above theory supposes that the orifice is indefinitely small, neglects the conditions and size of its sectional area, friction, the pressure of the atmosphere, and the resistance of the air to motion, which increases with the square of the velocity of the issuing fluid; the practical application that shows its discrepancies most strongly is the fact that the height of a jet is never equal to the head of pressure on it.

Third. The theory of flow, which is a combination of the two previous theories in a modified form, assuming

both uniform motion and the principle of gravitation, and is best expressed in the form of a formula—

$$V = (fg\,RS)^{\frac{1}{2}}$$

where V $=$ the mean velocity generated.

R $=$ mean hydraulic radius of the water section.

S $=$ the sine of the slope of the water surface.

This formula is a simple equation of the accelerating force of gravity down an incline with the retarding force of friction at any section at right angles to the course of flow, namely :—

$$gS = \left(\frac{V^2}{Rf}\right)$$

since, for uniform motion, the total accelerating force is equal to the total resistance.

This theory is the basis of calculation of flow in full tubes, and in open channels and unfilled pipes, where the principle still holds, though the equation should be strictly modified, the air above giving a resistance as well as the surface of the channel or tube below, though in a less degree.

However rigid these theories may appear in neglecting important points, they are yet generally true in the abstract, and no substitutes for them have yet been discovered; the consequence is that all hydraulic calculations are made to depend on them, their defects being made up by applying to them experimental coefficients, in preference to endeavouring to obtain theoretical accuracy by introducing into them niceties of theory that might fail in obtaining trustworthy results. It becomes, therefore, one of the important duties of a hydraulic engineer to apply these partly empirical formulæ with care and circumspection, especially guarding against taking for granted the formulæ and tabular results of different calculators, which vary in form and in result to a very great extent; some authors even

giving one-third more discharge than others as due to the same data. During practical work, again, time forbids a lengthy examination of principles; for this reason, therefore, this short chapter is given as an easy guide to the proper management and application to every-day wants of the working tables attached, which are based on the most improved modern principles.

2. NOTATION AND SYMBOLS.

To ensure clearness and rapidity of application of these theories, it is absolutely necessary that the nomenclature should be neither doubtful nor inconvenient, that the symbols be free from confusion, and the units of time, weight, and measurement, once adopted, generally adhered to as much as possible; this alone can cause the form of a formula to give at a glance any definite idea of the values of its terms and expressions.

The English foot has been generally adopted in this work as the unit of length, surface, and capacity, being the measure ordinarily used for heights and depths, as well as distances in survey work, and being now more capable of extended application than either the yard, link, or inch; the second has been generally taken as the unit of time, so that the numbers expressing discharges and velocities, which often are high numbers, may be as small as possible. This has been found to be perfectly manageable in practice. In the canal departments of Northern India the engineers have succeeded in abolishing chains, yards, and inches from their plans, estimates, and calculations, and in adhering generally to the second as a unit of time; they have also, on the Bari Doab Canal, adopted a mile of 5000 feet to the exclusion of the statute mile of

5280. This decimal system of measures, while retaining the use of a familiar unit, is found to save much needless labour in calculation, and at the same time has the great advantage of facilitating the conversion of foreign data and formulæ; the principal difficulties to contend with are the old habits of measuring water supply for towns in gallons instead of cubic feet, and of using dimensions of pipes in inches, instead of tenths of a foot; these obstacles will probably gradually disappear.

As regards the metrical system, although it is now adopted in all the civilized countries in Europe except our own, there seems little hope of its replacing our own measures to the entire exclusion of them for some time yet; hence it would not have been an advantage to have constructed the accompanying working tables on the metrical system, nor to have adopted it throughout this work in the data and formulæ; but as English engineers are now conversant with metrical measures, all such foreign formulæ and data mentioned are generally left in their original form, their conversion not serving any important purpose, but rather, on the contrary, causing complication needlessly.

Whether the decimal metrical system will hold its own for a very long time is yet a matter of considerable doubt: the number ten is not by any means in itself a convenient number for purposes of calculation, it is neither composed of a large number of factors, and hence admitting of easy subdivision, nor are its roots easily obtained; its use involves the necessity of referring to tables of logarithms in the greater part of the calculations made by engineers and scientific men; its sole advantage is a perfectly fortuitous one; it was chosen in ancient times as the first number to be represented by two digits, and the digital advantage it now possesses is perhaps its only one.

Should in the future any new notation come in vogue, which would readily enable the calculator to dispense with half of his logarithmic calculations, the advocates of the decimal system will then be looked upon as antiquated obstructors of progress.

For the present, however, the adoption of a decimal system seems absolutely inevitable, although it seems doubtful whether the English will first adopt one based on units familiar to them, or will change at once to the metrical system in its entirety.

The hydraulic engineer can, however, very conveniently adopt a decimal system based on the English foot for measures; nor apparently are there any very good reasons why the railway engineer should not do so also, except perhaps the tradition-loving habits of the multitude, and the meddlesome legislation in social matters under which we suffer, which enforces on him the adoption in Parliamentary plans of the whole of the old measures.

The advantage of adhering to one set of symbols in hydraulic formulæ, which sometimes appears very complicated, is sufficiently evident; with this view, therefore, the following general notation is drawn up, and the velocity notation of the Mississippi survey also attached for purposes of reference.

General Notation.

N = catchment area drained.

Q = discharge; q = discharge per square mile drained.

V = mean velocity of discharge, V_1, &c., other velocities.

V_x = maximum velocity in the cross section.

A = sectional area; a, a_1, a_2, subsidiary areas.

P = wetted sectional perimeter.

H = mean head or fall; h, h_1, h_2, subsidiary heads.

R = mean hydraulic radius = $\frac{A}{P}$

R_1 = prime hydraulic radius = $\frac{A}{P + W}$

S = hydraulic slope in terms of its sine = $\frac{H}{L}$;

thus $S = \frac{1}{500} = \cdot 002$ for a slope of 1 in 500.

L = a longitudinal length taken in the direction of flow; l, l_1, l_2, subsidiary lengths.

W = total transverse width, across the direction of flow; w, w_1, w_2, subsidiary transverse widths.

D = depth from surface level; d, d_1, d_2, subsidiary depths.

T = total time of discharge; t, t_1, t_2, subsidiary times.

f = experimental coefficient of fluid friction.

n = experimental coefficient for drainage discharges.

c = experimental coefficient for channel discharges.

m = experimental coefficient for orifice and overfall discharges.

g = velocity acquired by gravity in one second = 32·2 feet approximately.

All dimensions are generally in feet and decimals, and velocities and discharges are in feet and cubic feet per second.

Velocity Notation of the Mississippi Survey.

v = mean velocity of the river.

V = velocity at any point in any vertical plane parallel to the current.

$V_{100\,20}$ = velocity at a point 20 feet below the surface at a perpendicular distance of 100 feet from the base line.

U = velocity at any point in the mean of all vertical planes parallel to the current.

U_m = grand mean of the mean velocities in all vertical planes parallel to the current.

U_r = the mean of the bottom velocities in all such planes.

${}_{w_{,}}V$ = velocity at any depth below the surface at a perpendicular distance $w_{,}$ from the base line.

V_0 = velocity at the surface in any vertical plane parallel to the current.

V_2 and V_D = velocities at mid-depth and at the bottom in any such plane.

$V_{d_{,}}$ and V_m = the maximum and the mean velocities in any such plane.

---

b = sectional constant $= \frac{1 \cdot 69}{(R + 1 \cdot 5)^{\frac{1}{2}}}$.

l = length of a portion of river.

h = difference of level of the water surface at the two ends of l.

$h_{,}$ = the part of h consumed in overcoming longitudinal channel resistances, for a straight, regular course.

$h_{,,}$ = the part of h consumed in overcoming transverse channel resistances or irregularities.

W = river width at any given place.

w = perpendicular distance from the base line to any point of the water surface.

$w_{,}$ = perpendicular distance from the base line to the surface fillet moving with the maximum velocity.

D = total depth of river at any given point of surface.

d = depth of any given point below the surface.

$d_{\prime}$ = depth from the surface of the fillet moving with the maximum velocity in the assumed vertical plane parallel to the current.

m = depth from the surface of the fillet moving with a velocity equal to the mean of the velocities of all fillets in the assumed vertical plane parallel to the current.

Δ = maximum or mid-channel depth.

3. RAINFALL, SUPPLY, AND FLOOD-DISCHARGE.

All hydraulic works of irrigation, drainage, storage, water supply, river improvement, and land reclamation, are more or less affected by the amount and periodicity of the rainfall; for many of them careful and trustworthy rainfall statistics and data are absolutely essential; but the nature and amount of detail required vary with the nature of the work; works of storage being those that, perhaps, require the greatest amount of accurate information. In order that these local records should be sufficient to form a correct basis for the engineering data of these latter works, they should comprise observations extending over a period of ten years, or of the local period comprehending a cycle of rainfall from one season of maximum rainfall to another, including years of extreme drought; from these the following results can be deduced:—

1. The mean, maximum, and minimum monthly rainfall, from which the mean and extreme falls for each natural local season, wet, cold, and hot, can be obtained.

2. The mean and maximum daily falls, in twenty-four hours for each month.

3. Special occurrences, hourly falls, longest continuous falls and droughts.

These, arranged in a convenient tabular form, are all the rainfall data that the engineer will generally require.

In most cases, also, and especially in hot climates, evaporation records are also necessary; and sometimes, too, it is advisable to possess other meteorological data, such as those of humidity, temperature, atmospheric pressure, and wind; and, what is often difficult to procure, some data of absorption and percolation that would be applicable to the soils of the district under consideration.

On many of the works before mentioned, the first duty of the engineer is to account for the whole of the downfall, or to discover what becomes of it all, under both ordinary and unusual circumstances, so that he may be able to deal with more certainty of knowledge with that portion of it that more intimately affects his works; as, for instance, the bridge-builder with the floods, the engineer of storage works with the drought, and those of canals and river improvement with both. A geographical and geological knowledge of the catchment area, whose rainfall affects the works, is hence also needful; the boundaries of this area, its lines of watershed and drainage, its disposition as regards prevailing winds, the nature and porosity of its soil, and the amount of vegetation or cultivation on it, as well as any available records from which the quantities of water actually run off by its streams and rivers in various seasons may be arrived at, are all data necessary for establishing satisfactorily a perfect knowledge of the disposal of the whole of the rainfall under any circumstances.

In many instances it is, from want of sufficient information, utterly impossible to obtain this perfect knowledge;

in others, the deficient data may be supplied by approximations known to hold good in other similar cases, and a tolerably correct approximate balance struck between the downfall and the amount evaporated, absorbed, and run off; in any case, however, the engineer may, with time and means at his disposal, gauge the streams and rivers affecting his works, and make correct records of the amount of water run off in them at different seasons of the year, and in exceptional floods. Failing, however, both time and opportunity, such data have to be observed in a rapid manner that will enable him to determine this approximately; such as the section and fall of the rivers, the depths at various stages, floodmarks, and, if possible, a few velocity observations. The results principally required are the flood or maximum discharge, in cubic feet per second, of the river or stream draining the catchment area; its mean discharge throughout the year; and its minimum discharge in seasons of extreme drought, as well as in its ordinary low stage; dividing each of these by the number of square miles of catchment, similar results per square mile are obtained, which, when multiplied by 1·131, express the depth in feet of rainfall run off under each of those conditions. The relation between these quantities and the probable or approximate downpour over the catchment area can then be compared with those known to exist in other similar cases, and a valuable check on these important results thus obtained.

Flood Discharge.—The determination of the quantity of water discharged from a catchment area in a river or stream at a time of extreme flood, is a matter that is very often of the highest importance. Costly bridges have continually been sacrificed, and long lengths of canal damaged for want of sufficient attention having been paid to this subject.

When the data mentioned in the foregoing paragraphs can be obtained, and are properly handled, there is little difficulty in arriving at a correct result; but, as in many cases, only some of these are forthcoming, the bases of calculation are considerably narrowed, and various modes of obtaining a result necessarily varying with the available, have to be adopted.

When the catchment area has to be scaled from a map, and the highest maximum rainfall of 24 hours has to be taken from observations made at perhaps only one or two places near that area, the flood discharge may be approximately obtained by the equation,

$$Q = n \times 27 \sqrt{N},$$

where Q = flood discharge in cubic feet per second.
N = catchment area in square miles.
n = a local coefficient chosen with reference to the maximum day's rainfall of the place.

In using this as well as other formulæ of a similar type, records of flood discharges under somewhat similar condition are necessary for reference, in order that a practically correct value of n the coefficient may be assumed. This formula, which was originally adopted in connection with the inconvenient mode of estimating discharges in cubic yards per second or per hour, has very little to recommend it, the values of n being necessarily very wide in range; it still, however, has its adherents.

A more convenient one, having a narrower and more correct range of coefficients, is the following, which is a slight modification of that of Colonel Dickens, having a more extended application. It is

$$Q = n \times 100 (N)^{\frac{3}{4}},$$

and its terms are generally similar to those of the last formula. The values of n for India, generally lie between 1 and 24 : see coefficients at Table XII., Part 2, page lxx. of the Working Tables;—some further values of it applicable to various river basins in India, are also given in the tables of flood discharges at page [8] of the Hydraulic Statistics in the second part of this Manual. The values of the general expression, for a value of $n = 1$, are given for catchment areas of various sizes at pages ix. and x. of the Working Tables, Table IV., Part 1, and the chosen coefficient can be readily applied to these quantities.

The original form of this formula was simply $Q = 825 (N)^{\frac{3}{4}}$; it was considered applicable only to Bengal and Bahar in the first instance, and afterwards as applicable to all areas in the plains of India that have an annual rainfall of from 24 to 50 inches. It seems, however, more rational to use a variable coefficient depending on a similarity of general conditions, of which the maximum day's downpour is perhaps the most important. In Northern India where this latter is about 1·5 feet in or near hills, and 1·0 foot in the plains, the flood waterway allowed for bridges has generally been based on the assumption that the rainfall run off would amount to 1·0 foot in depth over the whole; and allowance has been made with these data for the flood waterway of the streams and rivers crossing both the Ganges Canal and the Sarhind Canal; in other cases, also, in Northern India, two-thirds of the depth of downpour is assumed to pass off in flood. It is, however, better to use the improved formula given and assume a coefficient suitable to similar conditions of catchment area.

A further attempt at arriving at a flood discharge by means of a formula has been made by Mr. Burge, Resi-

dent Engineer of the Madras Railway. His formula given in the Indian Professional Papers is

$$Q = 1300 \frac{N}{L^{\frac{2}{3}}}$$

where Q and N are as before, L = the length of the main river in miles, and 1300 is a coefficient applicable to a maximum downfall of 6 inches in 12 hours, and an area elevated from 500 to 1300 feet above mean sea level, consisting principally of unstratified rocks. It was deduced from observations on 27 bridges, of above 80 feet span, on the Madras Railway, and its results correspond closely with those of recorded flood sections; the errors lying between 4·64 feet too high and 3·40 too low in height of section. He argues most justly that the length of the river necessarily extends the time of the discharge, and hence diminishes the quantity passing off within a certain time; and that also the functions of discharge, the hydraulic slope, the cross section, and the head affected by the sinuosities in greater length, are reduced by it. Admitting this, the same principle would apply not only to the main river, but also to its tributaries; the number and conditions of the tributaries would probably be a more important consideration. Again, there is much difficulty in saying where a main river begins; so much so, that in the first place the introduction of an index of $\frac{2}{3}$ against a coefficient of 1300 would appear to be a needless attempt at exactitude; and in the second place the introduction of the length of the river at all in an equation of this sort is a matter incapable of very extended application; although in the instances from which this formula was laid down it has been very successfully introduced.

A better mode of introducing a function somewhat similar to this would be to apply in the equation the ratio of extreme breadth to extreme length of catchment

area; we have already a formula, the range of whose coefficients for India seem to be between 1 and 24—an important step already gained; and if this is modified into the form,

$$Q = n_{,} \frac{B}{L} 100 \, (N)^{\frac{3}{4}},$$

where B = extreme breadth of catchment area,
and L = extreme length of catchment area,
and $n_{,}$ = a new coefficient,
we obtain a more tangible improvement, capable of extended application. It is unfortunate, however, that for this formula a sufficient number of values of the new coefficient are not yet forthcoming; although in the instances in which it has been applied the improvement seems clearly manifested in reducing the range, so that for the present it is, perhaps, generally better to use that from which it is modified, while in special cases the ratio can be easily introduced.

Failing, however, such data as would be needful to enable one to choose a practically correct coefficient for formulæ of this type, it becomes necessary to fall back entirely on recorded flood marks, as a means of approximating to the flood discharge; and after gauging the discharge of the river in its ordinary stage, assume the flood discharge to be proportional to it according to the ordinary formula,

$$\frac{Q}{q} = \frac{A \sqrt{R}}{a \sqrt{r}}$$

where A is the sectional area up to flood mark, R its hydraulic mean radius, and a and r are similar quantities corresponding to the discharge (q) determined by observation.

4. STORAGE.

Reservoirs generally have for their object either the detention of flood water that might otherwise cause damage, as in works of river improvement, or the utilization of it in canals, of navigation, irrigation, or driving machinery, or for town supply. For the first purpose they must, to effect their purpose, be very extensive, and strongly aided by the natural formation of the country; for the last purpose they are, in one respect, excepting under very favourable conditions, particularly ill-fitted. The collection of drinking-water from the surface of land needs, in the first place, a clean, uncultivated and uninhabited tract of land as a catchment area; and in the second place, the water stored in the reservoir, which is liable to become putrescent, or seriously affected by the organisms, plants, and animalculæ that inhabit stagnant water, requires a very perfect and careful filtration, of a sort beyond the ordinary economic powers of municipalities or public companies. Indeed it is now asserted to be an uncontrovertible fact, that it is to the tainted water of rivers and reservoirs that one-half of most preventible diseases are due, the other half being caused by want of ventilation, faulty drainage, and mistaken modes of managing sewage, or, in other words, that impure air and tainted water are the chief enemies of human life; and there is, therefore, every reason to believe that in the future, when the general public become awake to this, and acquire enough energy to throw of the incubus of vested interests in the form of water companies, both tainted rivers and open reservoirs will be universally condemned as sources of drinking-water supply, and the water filtered, stored, and preserved against impurity by nature in the permeable

strata of the earth, will be drawn on in a more scientific and enlightened way than is at present usual, and be considered, as it justly is, a necessary of life. Another quarter of a century may show us scientific men objecting, on sanitary grounds, to the watering of our streets with such water as is now used in our food. It will therefore be only under very favourable conditions, or under circumstances that admit of no better alternative, that the water of storage reservoirs will be used to drink. For extinguishing fires, watering streets, and many other purposes, such water is, however, still valuable under ordinary circumstances.

The determination of the size and dimensions of a storage reservoir is a matter entirely governed by local circumstances and requirements. The assumptions that the area covered by it should bear a certain proportion to that of the catchment area, or that the amount of water stored should be as nearly as possible one-third of the available supply, are not by any means rules to be applied without a very large discretionary power, although there are rules laid down in various forms by different hydraulic engineers that very much resemble these. The object being the collection and retention of a certain amount of water for a definite purpose, and the circumstances being the local formation of the ground and the amount of downpour on the catchment area, all the economic considerations depend on these points.

The intention may either be to store as much water as possible within a certain amount of expenditure of cost, or only a definite amount sufficient for a certain purpose, or to store all that can possibly be obtained with a knowledge that the extreme amount would not be enough. Again, in one case, the quality of the water and the convenience of proximity, or of cleanliness of site, may be

considerations outweighing all others. If, therefore, the latter is the case, there are generally not many local conditions answering the purpose within which any choice can be made; and again, if a definite amount be required, the same may be generally said. It is only therefore in the case when the object is to store and utilise as much water as possible that much choice is left to the engineer.

Large artificial reservoirs being generally made on the natural surface of the ground, and bounded in one direction only by an embankment of earth, or a dam of masonry or brickwork, the first object is to choose a site or sites where the greatest amount of water can be stored with the shortest and least amount and length of embankment; for this purpose a river gorge, narrow and precipitous, terminating a great length of country, having a gradual fall towards it, offers the best ordinarily natural conditions; if, in addition, the lateral or transverse slope of the country is also very gradual, it becomes a large natural basin, with one narrow outlet; and if that admits of being easily dammed, an extraordinary advantage not often available presents itself.

The economy of constructing one large reservoir in preference to two or more small ones to hold the same amount would, perhaps, be evident at first sight to most people. The author has, however, met so large a number of persons that believe the contrary, that he is constrained to give the following mathematical proof of it by Graeff.

Let a single reservoir, or rather its contents when full, be supposed to consist of a number of laminæ, or layers of water, the sum of which will equal the total content, and let

K = the height of any one layer;

P and S = the perimeter and surface of its lower side;

P' and S' = the perimeter and surface of its upper side;

then the volume of this layer will be

$$= a\,K + \frac{b\,K^2}{2} + \frac{c\,K^3}{3}\ ;\ \text{where } a = S;$$

$$b = \frac{2\,P\,(S' - s)}{K\,(P' + P)}\ ;\ c = \frac{(S' - s)\,(P' - P)}{K^2\,(P' + P)}\ ;$$

Hence the above expression becomes

$$= \frac{K}{3\,(P' + P)}\left\{3\,S\,\overline{P' + P} + P'\,\overline{S' - S} + 2\,P\,\overline{S' - S}\right\}$$

$$= \frac{K.}{3\,(P' + P)}\left(P.\ 2\,\overline{S' + S} + P'\,\overline{2\,S + S'}\right).$$

In the case where the lateral and longitudinal slopes of the ground are uniform, we can imagine the reservoir to consist of one only of these layers; and its content will then represent that of the whole reservoir. In this case the height of the layer will be the extreme depth of water stored, and the quantities S and P will become indefinitely small in comparison with S′ and P′, and may hence be neglected: hence the total volume of water stored $= \frac{K\,S'}{3}$, and this is the volume of a reversed cone having S′ for its base; a demonstration that proves how rapidly the amount of storage increases with the depth of water, or with the height of the embankment.

To the height of dams, again, there is a practical limit: earthen dams of great height require an enormous section, being consequently very costly as well as dangerous, and are in themselves difficult to manage as regards escape; masonry dams have a limit to their height, due to the pressure per unit of surface on the foundation; the highest yet built does not exceed 164 feet, and it is very improbable that that height will be exceeded for some time to come, unless iron is made to enter largely into their construction.

After choosing a site for a proposed reservoir, one of the first points requiring attention is the determination

of its storage capacity up to different proposed levels of escape. For this purpose, marks are fixed at differences of level of about 5 or 10 feet, on any convenient short line of its section; and the contours of these levels marked out and surveyed all around the basin, in order to obtain the perimeters and areas at each contour, from which, as before shown, the contents of each lamina can be calculated, and the content up to any other contour. If, however, it be preferred to obtain this by means of a series of longitudinal and transverse sections taken up to the heights of the various contour levels, it is perhaps best to direct the former in conformity with the axis or axes of figure of the basin, and the transverse sections at right angles to them, and, as far as possible, at equal distances along them; although in many instances, unequal distances and inclined directions, more suited to the form and disposition of the ground, would give more correct results; and the inclined sectional areas, when multiplied by the cosine of the angle of obliquity, are easily reduced to the true values of their corresponding rectangular transverse sections. Should a winding river channel or depression form part of the basin, it is often more convenient and correct to estimate its content independently, and add it in afterwards.

The following are the three formulæ most used in obtaining the contents from the sectional areas:—

1. If there be only two sectional areas, A_1, A_2, taken at a time, at a common distance, d,

$$\text{the contents} = \frac{d}{2}(A_1 + A_2), \text{ or } = \frac{d}{3}(A_1 + A_2 + \sqrt{A_1 A_2}).$$

2. If there be three equidistant sections, A_1, A_2, A_3, taken at a time, and their common distance is d,

$$\text{the contents} = \frac{d}{3}(A_1 + 4A_2 + A_3) \text{ prismoidal.}$$

3. If there be any even number (n) of equidistant sections, A_1, A_2, &c., up to A_n, at a common distance, d, the contents $= d\left(\frac{A_1}{2} + A_2 + \&c.\ A_n - 1 + \frac{A_n}{2}\right)$.

The accuracy of result will of course depend on the closeness of the sections, and the suitability of their positions to the general form of the reservoir.

The capacity of the reservoir being obtained, the amount of supply that can be expected annually from the catchment area may be obtained, either in total quantities or in continuous quantities as cubic feet per second, by the aid of Parts 1 and 2 of Table II. of the Working Tables; in these calculations much labor is saved by deducting, in the first place, the allowance due to evaporation and absorption on the catchment area from the rainfall given, and making use of the available rainfall or rainfall run off as the basis of calculation for supply.

If a limited supply alone be required, the use of Part 1, Table III. of the Working Tables, will enable the contents of the reservoir, and extent of catchment area necessary to afford the supply to be rapidly determined. Part 2, Table III., may also be occasionally useful, where the supply is limited by the needs of an extent of land to be irrigated, or the population of a town requiring water for public purposes.

The section of waterway of escape has next to be determined; this depending on the flood discharge and the maximum downpour in twenty-four hours. In these calculations, Part 3, Table II. of the Working Tables is useful; so also are Parts 1 and 2, of Table IV., in connection with the formula already given for flood discharge.

The reduction or conversion of discharges or supplies

into either total or continuous quantities for various intervals of time, can be rapidly effected by the aid of the Table of Equivalents, Table XI., Parts 1 and 2; and their conversion into other measures, English or metrical, may be facilitated by the use of Parts 5 and 6 of the same table.

All these are of course simply modes of calculating, or of shortening the calculation, of the quantities of water; the determination of them has to be left to the discretion of the engineer and the requirements of the case. Should the supply be required to maintain a certain depth of water for navigation in a canal, the seasons, the supply deficient, the loss in the canals from evaporation and filtration, and all such data, will determine the amount;—if for irrigation, the amount of land, its quality of soil, and probable water duty; on this latter subject information is given in Chapter III. and in the Hydraulic Statistics, in Part 2 of this Manual, where data of the waterings and water duty usual in France, Spain, Italy, and Northern and Southern India, are given.

If, again, the supply is required either for motive power or the public purposes of town supply, the amount and height of delivery require determining with reference to local conditions; with reference to this, therefore, no guide would be of use. Lastly, if the object is the control of floods, the whole of the physical conditions of the river and its banks, from its highest watershed down to its mouth or embouchure in the sea, will be matters affecting the amount, and the management and regulation of the storage.

5. DISCHARGES OF RIVERS, OPEN CHANNELS, AND PIPES.

The various modes of gauging velocities and discharges are described in the chapter on field operations and guaging. The calculation of velocity or of discharges, under different conditions and for different data, may be considered independently of gauging. It is important to the engineer that he should at any time be able to calculate, in a few moments, the discharge of any pipe, river, or canal, from such data as he may possess.

The number of calculated velocity formulæ, their variety, and the wonderful amount of complication in them, as well as the want of exactitude of result they give, is truly astonishing; and when, on the other hand, one observes some engineers adhering slavishly to the tables and data of one hydraulician, others to those of another, and others again going through the conscientious, but very lengthy, course of examining everything that every hydraulician has said or done in the matter of calculation of mean velocity of discharge, one cannot but feel pained as well as surprised.

It would be quite out of place in this portion of a Manual of this description, which has for its object the supplying the engineer with information and tables for calculating his quantities and data in as rapid a way as practical correctness will allow, to enter into a detailed investigation of all these formulæ, and the reasons for setting them all aside, and adhering to that adopted in preference, and to the exclusion of all others; it will, therefore, suffice for the author here to mention the reason for adopting any one formula or conclusion as it is brought forward. A comparison of the results of various

hydrodynamic formulæ, will be given in Chapter III., among the miscellaneous detached paragraphs.

The general formula for discharge, based on the theories mentioned in the previous sections of this chapter, is

$$Q = A\,V = A\,(f\,g\;R\,S)^{\frac{1}{2}},$$

the terms of which are given in the general notation, page 10; the mean velocity of discharge being the smaller and more convenient quantity to deal with, for rivers and open channels, and the discharge itself being the quantity more often required for pipes, sewers, and closed tubes or tunnels of all sorts.

Taking, however, the expression for mean velocity of discharge, obtained by equating the accelerating effect of gravity down an inclined plane with the retarding effect of friction, it can be put into the form more convenient for English measures—

$$V = c \times 100\;(R\,S)^{\frac{1}{2}},$$

where c is a variable experimental coefficient, depending on the surface, the condition, the dimensions, and the hydraulic slope of the channel or pipe, and hence on a further experimental coefficient of fluid friction, and on a fresh development of the functions R and S: its value under extreme conditions varies from ·25 to about 2·00.

A correct formulated determination of the value of the coefficient, c, for all conditions, is a matter that can only be said to have been even approximately arrived at in the last few years, from an examination of the experimental results of d'Arcy and Bazin on the discharges of pipes, open channels, and ordinary rivers, and those of Humphreys and Abbot on the discharges of very large rivers; by Mr. W. R. Kutter, of Bern.

The determination of the coefficient, for which we are indebted to him, and tables rendering it easily found for

open channels and rivers of any sort or dimensions, in metrical measures, are given in his valuable articles in the "Cultur Ingenieur" for the year 1870.

From these the values of the coefficient suited to English feet and cubic feet per second have been reduced; they are given in the table for coefficients of all sorts, Table XII., under the head of coefficients of velocity of discharge, in Part 3, pages lxxi. to lxxx.: these are also further explained by the table of coefficients of fluid friction in Part 1, Table XII., page lxix.

With the aid, therefore, of these tables of coefficients, and the values of the expression 100 $(R\,S)^{\frac{1}{2}}$, given in Table VII., pages xviii. to xxv., the values of V, the mean velocity of discharge of rivers and open channels can be rapidly determined in a few moments, according to the most improved and correct method yet known.

With the aid of the same tables of coefficients and the values of the expression,

$$Q = c \times 39{\cdot}27\ (S\,d^5) \text{ when } c = 1,$$

given in Table VIII., pages xxvi. to xxxvi., the actual discharge of any full cylindrical pipe, sewer, or tunnel, can also be determined.

These tables, to which explanatory examples are attached, can also be used for the converse purpose of obtaining the head, diameter, hydraulic slope or hydraulic radius, due to given discharges of channels and pipes; it will, however, be necessary for the calculator to remember that all dimensions, even diameters of pipes, are invariably kept in feet, and that all slopes are kept in the form known as the sine of the slope, mentioned in the general notation, page 11, of this chapter. Should it be necessary to reduce these from gradients given in other forms, such as in feet per English mile, or as a fall

of unity to a certain length, Table VI., pages xiii. to xvii., will be found to save calculation.

So far for the velocity formula actually adopted, and the mode of working it in calculating results. As regards the formula itself, independently of the determination of the variable coefficient, it is none other but the Eytelwein formula, or Chezy formula, in a very much improved form, having the results of modern experiment incorporated with it. An examination of all the hydraulic formulæ for mean velocity shows that most, in fact, almost all of them, were modifications of the Chezy formula, some of them adding an additional term or function, and altering the value of the experimental coefficent, but still asserting its fixity. In the previous editions of this Manual, written before Mr. Kutter had published his valuable improvement, all these formulæ, having fixed coefficients, were rejected by the author, who at the same time asserted the principle that no fixed coefficient was suitable to all circumstances, and that the engineer should choose for himself a coefficient most suitable to the special circumstances, dimensions, and condition of the pipe, channel, or river, with whose discharge he was dealing; and that the results of experiments should be always consulted for this purpose.

A mode of successive approximation to the mean velocity was also recommended, first, assuming $c = 1$; and then from the mean velocity resulting, assuming a second value of c, according to the following table, a second true velocity of discharge was calculated.

v.	*c.*	*v.*	*c.*	*v.*	*c.*	*v.*	*c.*
1·0	·910	1·5	·960	2·0	1·000	2·5	1·023
1·1	·920	1·6	·968	2·1	1·005	2 6	1·026
1·2	·930	1·7	·976	2·2	1·009	2·7	1·030
1·3	·940	1·8	·984	2·3	1·014	2·8	1·033
1·4	·950	1·9	·992	2·4	1·018	2·9	1·037
						3·0	1·040

But these were intended to apply solely to canals in earth in good order. A few values of c, suitable to pipes under various velocities, were also given; but they were detached, and, from want of experiment, very insufficient. Yet the true state of the case, and the mode most advisable for adoption until investigations on a larger scale threw more light on the matter, was then clearly set forth.

Now that the experiments of d'Arcy and Bazin, of Humphreys and Abbot, and of Ganguillet and Kutter, have been comprehended in one formula, the labour of choosing a coefficient from experimental records is rendered entirely needless.

The determination or tabulation of this coefficient has gone through two stages of development. The first was that made by Bazin, based on the experiments conducted by d'Arcy, by Bazin himself, and by various engineers of the French Ponts et Chaussées, and is applicable to metrical measures. The principles asserted were that the coefficient depended on two quantities or qualities only, namely, the condition of surface of the bed and banks touched by the water, and the hydraulic mean radius of the section of discharge. Four categories of coefficients were adopted.

1st. For very smooth surfaces, well plastered surfaces in cement, and well planed plank.

2nd. For smooth surfaces, ashlar, brickwork, and ordinary planking.

3rd. For less smooth surfaces, as rubble.

4th. For earthen channels.

The values of the coefficient, A, being—

$$(1)\ 0{\cdot}00015\left(1 + \frac{0{\cdot}03}{R}\right)$$

$$(2)\ 0{\cdot}00019\left(1+\frac{0{\cdot}07}{R}\right)$$

$$(3)\ 0{\cdot}00024\left(1+\frac{0{\cdot}25}{R}\right)$$

$$(4)\ 0{\cdot}00028\left(1+\frac{1{\cdot}25}{R}\right)$$

and the corresponding value of c for the English formula of discharge being $=\frac{1}{100\sqrt{A}}$ for metres, and $\frac{1{\cdot}81}{100\sqrt{A}}$ for English feet; the French formula for metres being

$$\frac{RS}{V^2}=A,$$

and the English formula for feet being

$$\frac{V}{100\,(R\,S)^{\frac{1}{2}}}=c.$$

The values of these coefficients, adapted to the corresponding formula in English feet, are generally as follows, in their respective categories:—

R.	C. (1)	C. (2)	R.	C. (3)	C. (4)
1·	1·41	1·18	1	0·87	0·48
1·5	1·43	1·22	2	0·98	0·62
2·	1·44	1·24	3	1·04	0·70
2·5	1·45	1·26	4	1·06	0·76
3·	1·45	1·26	5	1·08	0·80
3·5	1·46	1·27	6	1·10	0·84
4·	1·46	1·28	7	1·10	0·86
4·5	1·46	1·28	8	1·11	0·88
5·	1·46	1·29	9	1·12	0·90
5·5	1·46	1·29	10	1·12	0·91
6·	1·47	1·29	11	1·13	0·92
7·5	1·47	1·29	14	1·13	0·95
8·	1·47	1·30	15	1·14	0·96
19·	1·47	1·30	18	1·14	0·98
20·	1·48	1·31	20	1·14	0·98

To obtain the values of coefficients of mean velocity

from the observed maximum velocity V_x, and values of R and S in English feet, we obtain from Bazin's formula $V_m = V_x - 14\sqrt{RS}$ for metres, which for English feet is

$$V_m = V_x - 23{\cdot}5\sqrt{RS},\ c = {\cdot}01\left[\frac{V_x}{\sqrt{RS}} - 25{\cdot}3\right].$$

Applying this coefficient to the formula $V_m = c \times 100\sqrt{RS}$, the true mean velocity of discharge V_m is obtained, and it is probable that this latter mode of determination is preferable both to the former and to the following method adopted by Kutter.

The second stage of development was effected by Kutter and Ganguillet; their own experiments on torrents and streams in Switzerland, combined with the results of Humphreys and Abbot on very large rivers, led them to believe that the coefficient should not be confined within so small a number of categories, and that also it was, besides being a function of the surface acted on by the water, and the hydraulic radius of the section, a function of the hydraulic slope.

They therefore extend the categories of coefficients suitable to open channels of all sorts in earthen beds into four distinct classes, and make some other additions to the categories adopted by Bazin; these new classes being ranged in accordance with the coefficient of fluid friction adopted as suitable to the surface under consideration.

A table of these general values of the coefficient of fluid friction is given in Part 1 of Table XII., page lxix.; and some local values from which the former were deduced by Mr. Kutter, are also given on the same page. The classes being determined by these means, the values of the coefficients of discharge are made to depend on them, as well as on the hydraulic slope and hydraulic radius of the open channel under consideration, and are

obtained for metrical measures by the following expression :—

$$c_{,} = \frac{23 + \frac{1}{f} + \frac{0{\cdot}00155}{S}}{1 + \left(23 + \frac{0{\cdot}00155}{S}\right)\frac{f}{R^{\frac{1}{2}}}}$$

which is also given in the following form :—

$$c_{,} = \frac{z}{1 + \frac{x}{R^{\frac{1}{2}}}}$$

where $z = 23 + \frac{1}{f} + \frac{0{\cdot}00155}{S}$ and $x = f\left(23 + \frac{0{\cdot}00155}{S}\right)$.

The reduction of this expression for application to English measures, for which $c = 0{\cdot}0181\, c_{,}$, is effected in pages lxxi. to lxxx. of the Working Tables; and if any convenient general value of f be assumed as applicable to the particular case, the coefficient corresponding to any ordinary values of R and S, likely to occur in practice on canals and rivers, can be read at sight.

The calculation of the discharge of pipes is conducted on exactly the same principle; although it is extremely unfortunate that the investigations of Ganguillet and Kutter were limited to open channels, and hence the application of his principles to pipes, though rationally superior to any other mode previously adopted, cannot be conducted with the same amount of experimental record in support, nor with the same amount of accuracy.

Assuming then the same formula for mean velocity of discharge—

$$V = c \times 100\,(R\,S)^{\frac{1}{2}},$$

and adapting it to terms of the diameter of a pipe in feet; it becomes for full cylindrical pipes and tubes of all sorts, where $R = \frac{d}{4}$

$$V = c \times 50\,(d\,S^{\frac{1}{2}}),$$

and as the actual discharge is the quantity more usually required direct in the case of pipes, this is—

$$Q = A\,V = c \times \cdot 7854\, d^2 \times 50\,(d\,S)^{\frac{1}{2}},$$
$$c \times 39\cdot 27\,(S\,d^5)^{\frac{1}{2}},$$

for discharges in cubic feet per second.

The converse forms of this expression being—

$$d = \frac{1}{c^{\frac{2}{5}}} \times 0\cdot 23 \left(\frac{Q^2}{S}\right)^{\frac{1}{5}},$$
$$H = \frac{1}{c^2} \times 0\cdot 0648\,\frac{Q^2}{d^5},$$

where H is the head in feet for a length of 100 feet, or is equal to 100 S.

The values of these quantities are given in Parts 1, 2, and 3, of Table VIII., for a value of $c = 1$, and the values of c given in the table of coefficients of discharge, Table XII., pages lxxi. to lxxiv., can be applied: the powers and roots of c can be taken from Part 7, Table XII.

With regard to these coefficients, it will be noticed that for want of sufficient experimental data, a coefficient of friction $f = 0\cdot 010$ has been assumed as applicable to enamelled or glazed metal pipes, and one of 0·013 for ordinary metal and earthenware or stone-ware pipes under ordinary conditions, but not new; and there is every reason to believe that these assumptions are generally correct, if we compare the smoothness of surface of a glazed pipe with that of very smooth plaster in cement, and that of an ordinary pipe, in average condition, with that of ashlar or good brickwork.

In applying however, to pipes the coefficients of discharge, resulting from the formula of Mr. Kutter, one would naturally be unwilling to push to extremes the principle, asserted by him as applicable to open channels, and would prefer stopping at a point where the experimental data now forthcoming leave us. It would, there-

fore, seem imprudent at present to assume the law of coefficients asserted by Mr. Kutter, to hold good for a hydraulic radius R less than 0·1 feet; which, for falls steeper than 0·001 give as a coefficient for glazed pipes 0·84, and for ordinary pipes 0·61. This limiting hydraulic radius of 0·1 feet is that of a 5-inch pipe, or a pipe having a diameter of 0·4 feet; and we therefore assume for the present, and until further investigation has thrown more light on the subject, that the coefficient of discharge for all full pipes, having a diameter less than 0·4 feet, will be the same as for those of that diameter.

The above-mentioned modes of calculating the discharge of rivers, open channels, and full cylindrical tubes, are intended to apply generally.

It will, however, be perfectly evident that this does not by any means preclude the application of an allowance or deduction made for special circumstances. In actual fact, few channels are either perfectly straight, perfectly regular, or free from lateral and longitudinal irregularities; these alone may affect the amount of discharge by as much as five per cent., even after making allowance for loss of head by bends and obstructions; and the local conditions of a river, the wind, the amount of silt in suspension, the motion of its shoals, the change of the set of its currents, all seriously affect a discharge calculated from data that make no allowance for these circumstances.

For canals and regular rectangular and trapezoidal channels in earth in good order, calculated discharges will naturally give results more correctly than for natural or river channels; the errors due to various irregularities being very much reduced. The formulæ of discharge are, however, as frequently used in determining the section of canal intended to convey a certain discharge, as to obtain a discharge from data of an actual canal.

In these cases, a consideration of the various forms of section, suitable to different purposes, is also necessary. This matter has been treated and repeated in nearly the same terms in all works on hydraulics, and there is, perhaps, nothing new to be said about it; the entire omission of it in a Manual of this description might, however, be liable to cause disappointment; and hence the following remarks, most probably based on the ideas of Eytelwein and d'Aubuisson, though, perhaps, taken through other channels now forgotten, are therefore inserted for purposes of reference.

6. THE FORM OF OPEN CHANNEL

that will give a maximum discharge, is that which, for a given sectional area, has the least wetted border or perimeter; the semicircle, like the circle, is geometrically known to possess this property, and regular demipolygons externally tangential to the semicircle, have also more or less this property, according as their sections more or less approximate to it in form; the semicircle, too, has its hydraulic radius equal to half its middle depth, and this also holds for trapeziums of maximum discharge.

Hence Neville's geometrical construction for determining the form of the trapezoidal channel of maximum discharge that has given side-slopes and sectional area.

From the middle of the top width of the proposed trapezium, describe a semicircle with a radius, equal to the proposed depth, and draw the given slopes and the bottom tangential to it.

This form gives the top width = sum of the side-slopes,
the mean width = half the perimeter,

and the area $= d^2 \left(\tan \frac{B}{2} + \operatorname{cosec} B\right)$,

where d = depth, and B = inclination of the slope with the horizon.

From these properties, the relative dimensions of trapezoids of maximum discharge, may be obtained for any side-slopes. They are given in the following table by Neville.

Relative Dimensions of Maximum Discharging Channels—(Neville).

		Factors for				
Slope.	Angle.	Depth.	Bottom.	Top.	R.	Area.
		In terms of the square root of the area.				
90°	0 to 1	·707	1·414	1·414	·354	$2d^2$
63° 26′	½ to 1	·759	·938	1·697	·379	$1·736d^2$
48° 34′	¾ to 1	·748	·675	1 996	·374	$1·784d^2$
45°	1 to 1	·740	·613	2·093	·370	$1·828d^2$
36° 52′	1⅓ to 1	·707	·471	2·357	·354	$2d^2$
33° 41′	1½ to 1	·689	·417	2·484	·345	$2·105d^2$
30° 58′	1⅔ to 1	·671	·372	2·608	·336	$2·221d^2$
26° 34′	2 to 1	·636	·300	2·844	·318	$2·472d^2$
Semicircle	curve	·798	·000	1·596	·399	$1·571d^2$
circle	curve	1·128	·000	·000	·282	$·785d^2$

These are most applicable in cases where heavy floods have to be provided for by a rapid drainage, and where the maximum discharge is the principal object.

For most practical purposes, however, such channels would be worse than useless, because depth is more expensive than width, because the high velocity generated might be destructive to the channel itself, and in cases where navigation is an object, the depth of draught would be too much affected by the fluctuation of supply; depth and velocity being thus limited, as well as the hydraulic slope, which is controlled by local circumstances, and the side-slopes, which depend on the nature of the soil, the width remains the only function of the section which admits of much variation.

Now, in a proportion of width to depth exceeding

14 to 1, which is about the lowest limit that will maintain a navigable depth, the side-slopes cease to remain a very important element, and the mean width can be dealt with equally well for rectangular and for flat trapezoidal sections; the practice in calculation, therefore, is, after assuming certain side-slopes, to reduce or increase the mean width by two or three feet at a time, until a safe bottom velocity is attained by the form of section thus approximated to, and the intended discharge thus arrived at. The next point is to know the relations between width and depth that give many sections that will discharge the same quantity with the same hydraulic slope. For this purpose their areas are inversely as the square roots of their hydraulic mean depths, and hence the square root of the cube of the channel sectional area, divided by the perimeter, must be constant. Thus:—

$$\left(\frac{w^3 d^3}{w + 2d}\right)^{\frac{1}{2}} = m,$$

and hence $d^3 - \frac{2m^2}{w^3} d = \frac{m^2}{w^2}$.

Solving which, we obtain for a value of $w = 70$, and for convenient values of d up to 6, corresponding values of m. Thus:—

d	·25	·50	·75	1·00	1·25	1·50	1·75	2·00
m	8·7	24·6	45·0	69·0	96·9	126	158	193
d	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0
m	267	349	437	531	629	732	839	951

This equation being also worked out for the same values of d and other values of w, the results are formed into a table of equal discharging channel-sections, given in Part 4, Table XI., page lx., which answers all practical purposes in determining dimensions of section for open channels of any size, by applying multiples and sub-

multiples to the dimensions there given. The table mentioned was taken from Stoddard's work, although there is also one very much like it in Neville's well-known work on Hydraulics, as there appeared to be no advantage in making a new one.

An additional table has, however, been made by the author to facilitate the determination of channels (not channel sections) of equal discharge, applicable to cases in which the variable coefficients of discharge, adopted by Mr. Kutter, are employed. Part 4, of Table XI., page lxi., gives a variety of depths, bottom widths, velocities, and hydraulic slopes, that are applicable to channels of one given discharge, and is useful in roughly determining dimensions and data necessary for various discharges.

The form of section of a pipe, with reference to its discharge, is a matter in which very little variation is practically possible: all small pipes being generally made cylindrical and kept constantly full. The quality of the interior surface of the pipe is however very important, the discharge being liable to be reduced as much as 33 per cent. by fouling and incrustation, the retarding influence being not so much the diminution of section as the increase of friction.

Formerly the method usually adopted in making allowance for incrustation consisted in reducing the diameter employed in calculating the discharge; the reduction being $\frac{1}{2}$ inch for pipes less than 3 inches in diameter, $\frac{3}{4}$ inch for 3-inch to 6-inch pipes, and 1 inch for pipes 6 inches to 1 foot and upwards in diameter. It is evident, however, that this principle is faulty, and that the reduction should be made for these circumstances in the coefficient of fluid friction employed in determining the coefficent of discharge. It is to be hoped also that in the

future water pipes will not be allowed to fall into the disgracefully filthy condition that has too often existed in England, and that some enamelling or glazing process, like that of Dr. Angus Smith, will be more universally adopted.

It will be evident from an examination of the original formula, that in order to obtain a maximum discharge from a pipe, its hydraulic mean depth, R, must be a maximum. A full cylindrical pipe, having $R = \frac{d}{4}$ seems at first sight to be nearly perfect in this respect; and, under high velocities, doubtless gives the greatest scouring power;—but the segmental circular section, leaving an upper section, whose angle is $78\frac{1}{2}°$ empty, admitting of the advantage of making the upper segment movable for cleaning, gives a maximum discharge for nearly filled pipes under smaller velocities, as thus shown :—

	Segmental.	Full Circle.
Hydraulic radius	·6	·5
Velocity	1·095	1·
Discharge	1·026	1·

The egg-shaped section, usually adopted for sewers, is good for intermittent unfilled pipes, as it fills higher and flushes better :—one form is generally adhered to, in which the diameter of the bottom circle is half that of the top, and the depth of the sewer, and the radius of each side curve, are each equal to once and a half the diameter of the top circle; they are generally calculated for filling to two-thirds of their depth, and in that state their discharges and velocities bear well-known proportions to those of cylindrical sewers :—viz.

	Velocities.	Discharges.
Cylindrical, full	1·00	1·00
Ovoid, ⅔-full	1·04	·89
Cylindrical, ⅔-full	... ...	·61

Calculations connected with pipes and sewers may be sometimes shortened by taking discharges through pipes of the same section in proportion to the square of the head, and through pipes of the same head proportional to the square roots of the fifth powers of the diameters. In these, Part 7, Table XII., is of use.

In dealing with the slopes of pipes, it must be remembered that the hydraulic slopes are those that are dealt with in all formulæ of discharge. Pipes are usually placed two or three feet below ground, to protect them from frost, and follow its sinuosities, rarely being allowed to rise above the mean hydraulic gradient or slope: should they do so, a great loss of head results, unless air vessels are applied at those points, from which the air is allowed to escape through cocks every two or three days. As again it is comparatively rare that a single pipe is laid to any very great distance with a uniform fall, being more generally cut up into lengths having different falls, it becomes necessary to proportion the diameter of the pipe in these different lengths, so that the discharge may be that due to the smallest diameter. When with such a series of pipes of different diameters the total head is given, and the discharge is required, the case does not admit of direct solution, as each pipe must have its own proper head; in this case it is best to assume a discharge, and obtain separate heads due to it for each pipe in the series; the true heads, both total and separate, may be then obtained by proportion, and the gradients of each pipe, as well as the mean hydraulic gradient of the whole series (which is the slope that would be adopted for a single uniform pipe throughout) marked on the section of the design. The final discharge can then be calculated from any one of the pipes. An example of this is attached to Working Table, No. X.

7. OTHER THEORIES OF FLOW.

Before quitting the subject of flow and entering into that of velocities, it may be as well to mention two apparently more perfect, though far less simple, theories of flow, which have not yet brought about sufficiently extended practical results in the determination of discharges. The first is that of Dupuit: it neglects friction on the sides of the section of flow, thus considering motion in all vertical planes to be the same, and dealing with horizontal laminæ only; the surface lamina is considered to be in the condition of a solid gliding over an inclined plane, and each lamina below, except the bottom one, is urged on by its own weight and its cohesion to the upper lamina; the bottom fillet is retarded by its adhesion to the bed. Putting this in the form of an equation, summing, rejecting certain terms, integrating and applying three numerical coefficients, Dupuit obtains a result, which for English feet is—

$$v = \frac{S.\ R\ A.}{\cdot 08\ W} - \cdot 082 + (\cdot 0067 + \cdot 9114\ R\ S)^{\frac{1}{2}}.$$

It is this formula that has produced more correct practical results generally, than any one of the formulæ having fixed coefficients: next to it, in order of correctness, coming the Chezy formula, with a fixed coefficient $c = 1$. This theory assumes that the uppermost lamina moves invariably with the maximum velocity, which is not the case; the neglect of the friction of the banks might again not vitiate results if applied to large rivers or shallow channels; it is probable, therefore, that a modification of calculation suited to the facts more recently discovered, about maximum velocity, might render this

theory very perfect as well as practical. For more information, refer to Dupuit's "Etude Théorique et Pratique sur le mouvement des eaux couraütes, Paris, 1848," and Claudel's Tables, which contain extracts therefrom.

The second theory is that of the Mississippi survey, mentioned in the Mississippi Report, Philadelphia, 1861, which deduces the new formula mentioned, as giving the most correct results of all yet known; it is, however, unfortunate in its formulæ being rather inconvenient in some respects. While, therefore, the investigation and deduction of the formula is valuable on account of the information, and results of experimental data applied to it, the result is not so useful as regards the practical use of the formula, which was virtually set aside by the Mississippi Survey, whenever careful river-gauging was carried out in favour of other equations deduced from velocity observation.

In a work of this scope, it is impossible to go beyond the mere outlines of the demonstration adopted. Adopting the notation of the Mississippi Survey given at pages 11 and 12, it may be stated as follows.

The theory accepts uniform motion and the usually accepted application of the laws of uniform motion, but in retarding force, denies the stability of position of maximum velocity, and makes allowance for the resistance of the air on the water surface, as well as for the effect of wind.

The process of reasoning follows through the following equations.

The equations obtained for the forces, are as follows :—

(1). $l G g a s = l (p + w) \phi \dfrac{U_o W + U_r p}{W + p}$

dividing both sides by Ggl,

putting $U_o = {\cdot}93v + ({\cdot}016 - {\cdot}06f)(bv)^{\frac{1}{2}}$

$U_r = {\cdot}93v + ({\cdot}06f + {\cdot}35)(bv)^{\frac{1}{2}}$

$$(2).\ \frac{as}{W+p} = \phi\left\{\cdot 93v + (bv)^{\frac{1}{2}}\frac{\left(W\left(\cdot 333 - \frac{d_1}{r}\right) + p\left(\frac{d_1}{r} - \cdot 667\right)\right)}{W+p}\right\}$$

putting $W = qp$, where q practically $= 1$ for large rivers.

$$(3).\ \frac{as}{W+p} = \phi\ (\cdot 93v + \cdot 167\ (bv)^{\frac{1}{2}} = \phi\ (z) = Cz^2.$$

$$(4).\ C = \frac{as}{(p + W)\,z^2}$$

by practical observation $C = \frac{s^{\frac{1}{2}}}{195}$, hence

$$(5).\ z = \left(\frac{195\,a\,s^{\frac{1}{2}}}{p + W}\right)^{\frac{1}{2}}$$

In this equation there are practically only four variables, a, $p + W$, s and z, once for ordinary natural channels p nearly $= 1\cdot 015$ W; hence if the values of any three are given, the fourth may be obtained, the transpositions of the equation being—

$$(6).\ s = \left(\frac{(p + W)z^2}{195a}\right)^2$$

$$(7).\ a = \frac{(p + W)z^2}{195\,s^{\frac{1}{2}}}$$

$$(8).\ p + W = \frac{195\,as^{\frac{1}{2}}}{z^2}$$

Now z is a variable, of which only two absolute values are known, viz., that for a rectangular cross section, and that for an ordinary river section, which are—

$$z = v + \cdot 167\ b^{\frac{1}{2}}\,v^{\frac{1}{2}}$$
$$z = \cdot 93v + \cdot 167\ b^{\frac{1}{2}}\,v^{\frac{1}{2}}$$

Substituting these in (5) and solving, we get for rectangular channels,

$$(9).\ v = \left(\sqrt{\cdot 0064b + (195r_1\,s^{\frac{1}{2}})^{\frac{1}{2}}} - \cdot 08b^{\frac{1}{2}}\right)^2$$

For ordinary river channels,

(10). $v = (\sqrt{\cdot 0081 b + (225 r_1 s^{\frac{1}{2}} - \cdot 09 b^{\frac{1}{2}})^2}$;

For large rivers, where $r > 12$ feet, and where $b = \dfrac{1 \cdot 69}{(r + 1 \cdot 5)^{\frac{1}{2}}} = \cdot 1856$, the first term may be neglected, and this latter equation becomes—

(11). $v = ([225 r_1 s^{\frac{1}{2}}]^{\frac{1}{4}} - \cdot 0388)^2$.

If the discharge is known, and also two of the four variables in equation (5), provided they are not a and v, the other two variables may be computed by eliminating the unknown variable in the second member of that one of the transpositions of equation (11) whose first member is the variable sought, by substituting for it its value deduced from the equation (12),

$$v = \frac{Q}{a}.$$

No difficulty will be found in performing the calculation, except when s and $p + w$ are the known variables, in which rate an equation of a higher degree than the second cannot be avoided, and successive approximation must be adopted as follows:—

Assume a value of a, and find two values of v, one from equation (12), the other from (10) or (9), as the case may require; these values of v will not agree, hence assuming a new value for a, until the resulting values of v are identical.

An application of the above-mentioned Mississippi formulæ to the discharges of canals, or even of small streams and rivers, cannot by any means be considered satisfactory as regards result; although for large and very large rivers, the amount of exactitude resulting may exceed that of any other known formulæ.

8. VELOCITIES IN PIPES AND ARTIFICIAL CHANNELS.

The laws of the distribution of velocity in the section of an open channel, canal, or river, are not yet satisfactorily determined. A certain amount of knowledge has been deduced from observation of the variation of velocity in the vertical planes, but as regards that in the horizontal planes of the section, nothing has absolutely—and very little relatively—yet been determined. In pipes, on the contrary, the conditions of velocity are comparatively simple. All the valuable information on this subject, quoted in this work, is that deduced by d'Arcy and Bazin, and by Humphreys and Abbot, from the results of their extensive experiments.

The experiments of d'Arcy, in 1851, established the following law of velocity in full pipes :—

$$\frac{V - v}{\sqrt{RS}} = 11{\cdot}3\left(\frac{r}{R}\right)^{\frac{3}{2}},$$

This equation is in terms of metrical measures—

V = central velocity.

v = the velocity anywhere at a distance $= r$ from the centre.

R = the radius of the pipe.

S = the loss of head or slope per running metre.

This equation in another form becomes—

$$V - v = \frac{11{\cdot}3}{R} r^{\frac{3}{2}} \sqrt{S}.$$

This formula was deduced by d'Arcy from observations taken at from one-third to two-thirds of the radii of various pipes from the centre; beyond $\frac{2}{3}$ of the radius, it is probable that the law does not hold good, and that the decrement of velocity should be more rapid than that

indicated by the formula. Under any circumstances, however, it is clearly established that the velocities in a full cylindrical pipe, are equal at all points equidistant from the centre, and that the above law of decrement holds good for the central $\frac{2}{3}$ of the diameter taken in any direction. In a pipe of rectangular section, the velocities are equal at any four points, taken symmetrically with reference to the centre of figure.

In open channels, however, this almost mathematical accuracy is entirely absent, and the perturbations produced near the surface of the water does not allow us to hope that any formula can be arrived at, which would give the actual velocity at any point in terms of the mean velocity and the co-ordinates determining the position of that point. These perturbations appear to be more considerable in proportion to the diminution of velocity, and the increase of depth of channel, and are coincident with a depression of the locus of maximum velocity; in the extreme cases, the curves of equal velocity in the section cut the surface of the water very obliquely.

The following are the conclusions drawn by Bazin on this subject:—

1st. For a very wide rectangular channel—

$$\frac{V - v}{\sqrt{HS}} = K \left(\frac{h}{H}\right)^2,$$

where V = central velocity at the surface.

v = velocity at a point at a depth h below it.

H = total depth of water.

S = hydraulic slope of the water surface.

The above law of velocity is proved to hold good for very wide channels; the cases under experiment give a practically constant value of K = 20·0, the extremes varying between 15·2 and 24·9;—it would also appear that for a

rectangular canal of infinite width, in which the influence of the sides was entirely made to disappear, K would $= 24{\cdot}0$.

When, however, the depth of a rectangular channel is great enough, in proportion to the breadth, to make the influence of the lateral walls show itself in the middle of the current, this law does not hold, nor does any law of decrement of velocity seem possible, and mere generalisations, in terms of the mean velocity, can alone be arrived at.

If, then U = the mean velocity in a canal, the section of which is very great in proportion to its depth—

$$U = \frac{1}{H}\int_{0}^{H}\left[V - K\left(\frac{h}{H}\right)^{2}\sqrt{RS}\right]dh$$

$$= V - \frac{K}{3}\sqrt{RS}$$

and the depth h below the surface is determined by the expression $\left(\frac{h}{H}\right)^{2} = \frac{1}{3}$; whence $h = 0{\cdot}577$ H, which is, in fact, saying that the mean velocity is found at about $\frac{3}{5}$ of the total depth. This, however, assumes the before-mentioned parabolic law of the decrease of velocity in each vertical plane, a hypothesis only admissible in a very large and perfectly regular canal.

In fact, however, and from experiments quoted, it appears that the locus of mean velocity is often below $\frac{3}{5}$ of the depth, and more often below $\frac{3}{4}$ of it; and that when the depth of the canal is great, and the velocity feeble, the curve of mean velocity approaches still nearer the bottom, and goes as low as $\frac{4}{5}$ of the depth.

Taking the above relation $U = V - \frac{K}{3}\sqrt{RS}$, where $\sqrt{\frac{RS}{U}} = \sqrt{A}$, and $K = 24{\cdot}0$, for a channel of infinite width; in this case also we get $\frac{V}{U} = 1 + 8\sqrt{A}$, as a result applic-

able to this special case, which supposes the parabolic law applicable throughout the whole breadth of the channel; and this differs greatly from the results of experiment on channels, which gives $\frac{V}{U} = 1 + 14\sqrt{A}$.

The locus of maximum velocity is, however, not always at the centre of the surface, but is at a greater depth in proportion as the depth of the canal is greater and the mean velocity is less, being sometimes as low as $\frac{2}{5}$ the total depth.

The determination of bottom velocity can, in rectangular canals, be alone made in the special case of one supposed to be of infinite breadth: for this case, putting $h = H$ in the original formula, we obtain the velocity $w = V - K\sqrt{RS}$; but in all other cases no law can be given. The greatest of bottom velocities is in the middle and the least at the sides.

The velocity along the vertical sides of a rectangular canal, is generally greater in the middle than at the top or at the bottom; but beyond this fact, the determination of the exact velocity at any point of the side remains a very difficult problem yet unsolved.

The laws of velocity in canals of semicircular section are far less complicated than those of rectangular section:— the law of decrement of velocity is expressed in the following formula:—

$$\frac{V - v}{\sqrt{RS}} = 21\left(\frac{r}{R}\right)^3$$

the extreme values of the coefficient deduced from experiment being 18·2 and 23·2; and the terms of the expression being similar to those in the equation for decrement of velocity in sections of pipes before mentioned:—

If in this we make $r = R$, we obtain as for rectangular channels, the bottom velocity, $w = V - 21\sqrt{RS}$.

And the mean velocity will be deduced thus :—

$$U = \frac{1}{\pi R^2}\int_0^R \left[V - K\sqrt{RS}\left(\frac{r}{R}\right)^3\right] 2\pi r.\,dr$$

$$= V - \tfrac{2}{5}K\sqrt{RS}; \text{ where } \frac{\sqrt{RS}}{U} = \sqrt{2A};$$

$$\text{hence } \frac{V}{U} = 1 + \tfrac{2}{5}K\sqrt{2A}; \text{ where } K = 21$$

$= 1 + 11{\cdot}9\sqrt{A}$: an equation differing but little from that deduced from experiment on semicircular canals.

The radius $r_{,}$, of the circle of mean velocity of the section $= R.\sqrt[3]{\tfrac{2}{5}} = 0{\cdot}737$ R ;—which is saying that this is at about three-quarters of the radius from the centre, whereas in fact it is farther.

Taking finally the two expressions for decrement of velocity in canals of rectangular and semicircular section,

$$\frac{V - v}{\sqrt{HS}} = K\left(\frac{h}{H}\right)^2 \text{ and } \frac{V - v}{\sqrt{RS}} = K\left(\frac{r}{R}\right)^3$$

a general expression may be deduced from them,

$$V - v = \phi\sqrt{RS};$$

and as under these circumstances absolute velocities cannot be dealt with, it is better to make use of relative velocities, and by dividing each side of the general equation by U to transform it into the form

$\dfrac{V - v}{U} = \phi\sqrt{A}$; which is therefore true for all canals

where ϕ is a function of the relative (not of absolute) co-ordinates determining the position of the point whose velocity is under consideration, their values being taken in proportion to the dimensions of the section.

With regard to velocities in natural and artificial

channels generally, by far the most important result arrived at by d'Arcy and Bazin, is the relation between the maximum velocity and the mean velocity of discharge, represented by this equation, suitable to metres:

$$\frac{V}{U} = 1 + 14\sqrt{A}; \text{ and since } A = \frac{RS}{U^2}; V - U = 14\sqrt{RS};$$

these equations reduced to English measures become

$$\frac{V}{U} = 1 + \frac{25{\cdot}34}{c \times 100}; \text{ and } V - U = 25{\cdot}34\sqrt{RS}.$$

The advantage derived from the application of this law in gauging is probably greater than that of any other velocity discovery of modern times.

Velocities in Natural Channels.

The laws of variation of velocity in horizontal planes, with reference to different forms of section have not yet been satisfactorily deduced, such velocities have therefore to be determined locally when required; the horizontal curves of velocity again vary much in different stages of the river or stream under consideration; the records therefore of such velocities involve much labour, and have not yet shown themselves of sufficient practical importance to repay the labour and trouble of their observation. The laws of variation of velocity in vertical planes have been most fully investigated by Captains Humphreys and Abbot on the great Mississippi Survey.

It was previously generally believed that the maximum velocity of any river or channel was that on the surface in the middle; that the mean velocity varied between ·7 to ·95 of the maximum velocity, in natural channels, and was generally ·8 for rectangular sections; and that the bottom velocity equalled twice the mean velocity less the maximum velocity, or ·6 of the maximum velocity for rectangular sections. There were also numerous other

equations of relation between these quantities given by various theorists, none of them probably more correct than the above.

There is every reason to believe that this subject, difficult in itself, has been rendered more difficult to manage from the falsification of results by using many different complicated instruments, possessing inherent errors, and not admitting of a just comparison; the Mississippi observations being conducted on a very large scale, and in the simplest manner possible, have brought forth very important results. From their experimental data it has been deduced that the velocities at different depths below the surface in a vertical plane, vary as the abscissæ of a parabola, whose axis is parallel to the water-surface, and may be considerably below it, thus proving the maximum velocity to be generally below the surface; the equation of this curve with reference to its axis, taking the depths, relatively to the total depth, as ordinates, was obtained in the form—

$$y^2 = 1{\cdot}2621\ D^2 x$$

where D = total depth of bed below the surface, and x and y are the co-ordinates to the axis.

They also deduced that if d is the depth of the axis of the parabola, or locus of maximum velocity from the surface, then

$$d_1 = ({\cdot}317 + {\cdot}06 f)\ R$$

where R = hydraulic mean radius, and f = force of wind taken positive or negative, and taken = 1 when the velocity of the wind and current are equal, and = 0 for a cross wind or calm.

The following are other important equations, with regard to velocity in vertical planes, that they deduced, which though they are not so useful practically as might be wished, are inserted here for reference.

(For symbols refer to the notation given in the paragraph on that subject.)

Formulæ for velocity in any vertical plane:

(1) $b = \dfrac{1 \cdot 69}{(D + 1 \cdot 5)^{\frac{1}{2}}} = \cdot 1856$ only when D ⁊ 30 feet.

(2) $d_1 = (\cdot 317 \times \cdot 06 f)\, D$ very nearly.

(3) $V = Vd_1 - (bv)^{\frac{1}{2}} \left(\dfrac{d - d_1}{D}\right)^2$

(4) $V_o = Vd_1 - (bv)^{\frac{1}{2}} \left(\dfrac{d_1}{D}\right)^2$

(5) $V_D = Vd_1 - (bv)^{\frac{1}{2}} \left(1 - \dfrac{d_1}{D}\right)$

(6) $V_m = \frac{2}{3} Vd_1 + \frac{1}{3} V_D + \dfrac{d_1}{D} \left(\frac{1}{3} V_o - \frac{1}{3} V_D\right)$

(7) $V_{\frac{D}{2}} = V_m + \frac{1}{12} (bv)^{\frac{1}{2}}$

(8) $V_{d_1} = V_m + (bv)^{\frac{1}{2}} \left(\frac{1}{3} + \dfrac{d_1 (d_1 - D)}{D^2}\right)$

(9) $V = V_m + (bv)^{\frac{1}{2}} \left(\dfrac{D\left(\frac{1}{3} D - d_1\right) + (2\, d_1 - d)}{D^2}\right)$

in which equation (9) is a mere combination of equations (3) and (8).

For velocity in the mean of all vertical planes the following formulæ have been deduced:

(1) $b = \dfrac{1 \cdot 69}{(r + 1 \cdot 5)^{\frac{1}{2}}}.$

(2) $d_1 = (\cdot 317 + \cdot 06 f)\, r.$

(3) $U_m = \cdot 93 v.$

(4) $U = \cdot 93 v + \left(\dfrac{dr(\cdot 634 + \cdot 12 f) - d^2}{r^2} - \cdot 06 f + \cdot 016\right)(bv)^{\frac{1}{2}}.$

(5) $U_o = \cdot 93 v + (\cdot 016 - \cdot 06 f)\, (bv)^{\frac{1}{2}}.$

(6) $U_r = \cdot 93 v\, (\cdot 06 f - \cdot 35)\, (bv)^{\frac{1}{2}}.$

(7) $Ud_1 = \cdot 93 v + \{[\cdot 317 + \cdot 06 f]^2 - \cdot 06 f + \cdot 016\}\, (bv)^{\frac{1}{2}}.$

(8) $v = \left([1 \cdot 08\, U_{\frac{r}{2}} + \cdot 002 b]^{\frac{1}{2}} - \cdot 045 b^{\frac{1}{2}}\right)^2.$

The most important result of all these data and deduc-

tions is the following, a fact of great practical use in gauging rivers, that the ratio of the mid-depth to the mean velocity in any vertical plane is independent of the width and depth of the stream (except for an almost inappreciably small effect) absolutely independent of the depth of the axis of the curve before referred to, and nearly independent of the mean velocity The formula expressing this is

$$(7)\ V_{\frac{D}{2}} = V_m + \frac{(bv)^{\frac{1}{2}}}{12}, \text{ where}$$

V_m is the mean velocity on any curve in the vertical plane.

$V_{\frac{D}{2}}$ is the mid-depth velocity.

v is the mean velocity of the river.

D is the depth of the river at the spot.

$$b = \frac{1{\cdot}69}{(D + 1{\cdot}5)^{\frac{1}{2}}}, \text{ which when } D > 30 \text{ ft.} = {\cdot}1856.$$

The application of this result to gauging is shown in Chapter II. on Field Operations.

9. BENDS AND OBSTRUCTIONS.

The irregularities of a river materially affect its velocity; the following remarks on this subject, by Captains Humphreys and Abbot, are instructive on this point.

"Even on a perfectly calm day, there is a strong "resistance to the motion of the water at the surface, "independent of, and not mainly caused by the friction "of the air; the principal cause being a loss of force, "arising from the upward currents or transmitted motion "caused by the irregularities at the bottom. There is "also an almost constant change of velocity at various

"depths, resulting from the wind in a great measure; "and eddies changing their position and magnitude, "cause variations in the velocity of the river at a given "point, and these again are influenced in intensity by "the wind."

Such irregularities are of course beyond calculation: others again may, in some instances, have their results approximated to, and allowances made for them, by considering a certain portion of the head on the stream as neutralized by them; and these are known as bends or obstructions whose effects are within the range of calculation. Generally the disturbing effects of lateral bends and curves, and of shoals and obstructions, constituting vertical bends, as well as alterations of section, cannot be calculated with any practical accuracy. It is, therefore, best entirely to avoid such difficulties; but when this cannot be done, the following formulæ may be used in preference to neglecting to make any allowance at all.

The general formula for loss of head, h, due to bends in rivers, canals, or pipes, is—

$$h_{,} = \frac{m.\ n.\ Sin^2 a.\ V^2}{\sqrt{R}}$$

where m is, an experimental coefficient, generally taken $= \cdot 5184$;

n = the number of bends for which allowance is to be made;

$Sin a$ = the sine of the angle of one bend, which should not exceed 90°;

h and R are in feet, and V is in feet per second.

A more modern formula suited to rivers, is that adopted by the Mississippi Survey, it is—

$$h_{,} = \frac{V^2\ Sin^2 a}{134};$$

where a = angle of incidence of the water in passing

round the bend ;—it is, however, always assumed that each angle is one of 30°, and the effect is estimated as due to the number n of such bends or deflections of 30°; and this enables the formula to be put into the simpler form—

$$h_{,} = \frac{n\ V^2}{536} = n\ V^2 \times 0{\cdot}001865.$$

The values of this formula, for various velocities and bends, are given in Part 2, of Table X., page li., and an explanatory example at page lii.

A formula more suited to bends of pipes, is that of Weisbach; it is for cylindrical pipes—

$$h_{,} = \frac{a}{180^{\circ}}, \frac{V^2}{2g} \times \left\{ {\cdot}131 + 1{\cdot}847 \left(\frac{r}{R}\right)^{\frac{7}{2}} \right\}$$

and for rectangular tubes—

$$h_{,} = \frac{a}{180^{\circ}}, \frac{V^2}{2g} \times \left\{ {\cdot}124 + 3{\cdot}104 \left(\frac{d}{2R}\right)^{\frac{7}{2}} \right\}$$

but as the bends of pipes, known as quarter bends, are generally taken as 90°; the factor—

$$\frac{a\ V^2}{180^{\circ} \times 2g} \text{ becomes } = \frac{V^2}{128{\cdot}8} = {\cdot}007764.$$

In this formula r and R are the radii of the pipe and of the bend, and the other terms are as before. The loss of head due to bends in pipes is, however, generally required as corresponding, not to mean velocities of discharge, but to the discharges themselves. The values given by this formula have, therefore, been tabulated in this form, and are given in Part 1, of Table X., page l.; an explanatory example is also attached.

The ordinary formula for calculating the rise in feet resulting from an obstruction in the section of a river channel, is that of Dubuat; it is—

$$h_{,,} = \left(\frac{V^2}{m^2 . 2g} + S\right) \left\{ \left(\frac{A}{a}\right)^2 - 1 \right\}$$

where A, a, are the normal and the reduced sectional areas,

S is the sine of the hydraulic slope of the river, and m is the experimental coefficient.

Now, as in most cases, S is less than ·001, that term may be neglected, and taking $m = \cdot 96$, $m^2 = \cdot 92$, and the formula becomes—

$$h_{\prime\prime} \doteq 0{\cdot}0169\, V^2 \left\{ \left(\frac{A}{a}\right)^2 - 1 \right\}$$

The values of this are given in Part 3, of Table X., page li., and an explanatory example on page lii.

10. DISCHARGE FROM ORIFICES AND OVERFALLS.

The discharge from orifices and overfalls, which to the hydraulic engineer generally resolve themselves into sluices and weirs, is a subject that was fully entered into by hydraulicians of past times, and to which very little information has been added by recent experimentalists. Nor is it by any means likely that further contributions will be soon made to this branch of hydraulic science, as there have recently been to that of the discharges of open channels; the practical interest attaching itself to the exact determination of discharge of a sluice or a weir, not being in excess of the amount of exactitude already attained. All accepted information on this subject being to be found, with but little variation, in the older books, the author has had little choice left to him, and has therefore taken the following notes almost entirely from Bennett's translation of d'Aubuisson's hydraulics.

Setting aside the experiments of the more ancient

philosophers, and assuming that the discharge from any orifice is

$$Q = AV = A.\, m \sqrt{2g\,H}$$

where H = the head of pressure of the orifice,

m = the coefficient of reduction obtained by experiment,

V = the mean velocity of discharge,

and the pressure being supposed to be kept perfectly constant, the first of the more modern hydraulicians to obtain experimental values of m, on a scale larger than the previous very petty experiments, was Michelotti. His experiments conducted at Turin in 1767, under heads of pressure up to 22 feet, determined coefficients of reduction varying from 0·615 to 0·619, for circular orifices, up to 6½ inches in diameter, and coefficients varying from 0·602 to 0·619 for square orifices, up to 3 inches in length of side. The next important experiments did not so much include increase of head as increased dimension of opening. Messrs. Lespinasse and Pin, Engineers of the Languedoc Canal, 1782 to 1792, made experiments on rectangular openings, or sluices 4·265 feet broad, and having heights varying from 1·575 to 1·805 feet, under heads on their centres of, from 6·2 to 14·5 feet; the coefficients deduced varied from ·594 to ·647, the mean being 0·625; they also observed that the discharge from two sluices opened at one time side by side, was not double that from one sluice. The next important experiments were those of Poncelet and Lesbros, at Metz, in 1826; they deduced a law for the determination of coefficient of discharge of rectangular orifices under various proportions of head of pressure and depth of opening to width; these coefficients reduced by Rankine are given in a tabular form in Part 4 of Table XII., at page lxxxii. of the working tables. The next important experiments recorded were those conducted

by M. George Bidone, at Turin, in 1836, on orifices on parts of which the contraction was suppressed, the extreme of suppression being a case in which the whole of the contraction was suppressed by fitting an interior short tube to the mouth of the orifice: his resulting formula of discharge was for rectangular orifices—

$$Q = m\,A\,\sqrt{2gH}\,(1 + 0{\cdot}152\,\tfrac{n}{p})$$

and for circular orifices,

$$Q = m\,A\,\sqrt{2gH}\,(1 + 0{\cdot}128\,\tfrac{n}{p})$$

where n is the portion of the perimeter p, whose contraction is suppressed.

About this time also some further experiments were made by Castel and d'Aubuisson; and some by Borda on orifices in sides not plane.

The results of all these experiments show that the extreme limits of the value of m, are 0·50 and 1·00 for orifices in all sorts of sides, and under all conditions, and are 0·60 and 0·70 for orifices in plane sides: also that the general mean value of m for orifices in a thin plate is 0·62; this, however, is perhaps more true for small circular orifices than for any other class of them. In this case therefore

$$V = 0{\cdot}62 \times 8{\cdot}025\,\sqrt{H} = 4{\cdot}975\,\sqrt{H},$$

and for rectangular orifices of a similar class, the values of m, ranging from 0·572 to 0·709 given at page lxxxii., must be applied to the general formula

$$V = m \times \sqrt{2gH}$$

in order to determine the mean velocity of discharge, which when multiplied by the sectional area gives the quantity discharged per second.

In the special case in which the reservoir of supply, still being kept at a constant level, is seriously affected by

the velocity of the water supplying it, the discharge of the orifice will be augmented on this account, and then

$$V = m\sqrt{2g\left(H + \frac{W^2}{2g}\right)} = m\sqrt{2gH + W^2},$$

where W = the initial velocity of entrance.

For the special cases in which an open canal is attached to the orifice at its exit, in such a manner that the sides and bottom of the canal are continuations of those of the orifice, the coefficient of contraction remains the same, except when the head on the orifice is less than $2\frac{1}{2}$ times the height of the orifice: in this latter case the coefficient may have to be materially reduced. An extreme case given by Poncelet and Lesbros, being one of a discharge through an orifice 0·164 feet high, under a head of 0·118, gave a value of m = 0·452, while without an attached channel the value of m was = 0·612: further, when the level of the attached channel was exactly at the same level as the floor of the reservoir of supply, the value of m was reduced to 0·443. The law of reduction of coefficient necessary for these cases is not yet given in a definite form.

The inclination of the attached channel, when less than one in 100 did not affect the coefficient in any way, but when increased to one in 10, had the effect of increasing the coefficient from 3 to 4 per cent.

The above includes all the general deductions about orifices that are likely to be of any use to the engineer; a more practical collection of coefficients of discharge for orifices is given in Part 4 of Table XII., at pages lxxxi. and lxxxii.; and the value of the expression

$$V = m\sqrt{2gH}$$

is given, for various heads, and for all the values of m that are commonly used in Table IX., pages xxxvii. to xlviii.; some explanatory examples also follow that table.

It may be observed, however, that although the minutiæ of discharges under certain experimental conditions have been sedulously preserved, there is yet considerable doubt what coefficient should be used for the larger sluices or openings that occur in practice. It is no doubt unfortunate that experimentalists should differ, but at the same time the circumstances under which the amount of discharge from a sluice is an important consideration only occur generally to those who are capable, and have the opportunity of determining it accurately by experiment themselves.

The ordinary coefficient for a sluice of moderate size, for small lock or dock-gates, or mill-gates, is generally taken at 0·62: that for a narrow bridge-opening, which may be considered as a large sluice, at 0·82; and that for very large well-built sluices, large wide openings out of reservoirs continuing at a level with the bottom of the reservoir, and large bridge-openings of the modern type, at 0·92.

The term H, representing the effective head of pressure, is differently estimated in various cases: in ordinary cases of sluices, supplied from a reservoir above them, the head is the difference of level between the surface of the water in the reservoir and the centre of figure of the sluice; but when the sluice is drowned, that is, has a perceptible depth of water standing below its exit, but above the sluice itself, the head is the difference of level of the water above and of that below it; in bridge-openings also, the head is the difference of water level above and below the bridge.

The most recent experimental determination of coefficients of discharge for head-sluices supplying small channels is that of d'Arcy and Bazin; the results of these operations will be given with the account of the mode of gauging adopted by them in Chapter II.

Orifices with mouthpieces attached were even in the time of the Romans known to have a greater discharge than without them. In order to effect this increase it is, however, necessary that the length of the attached or additional tube should be twice or three times the diameter of the orifice, otherwise the fluid vein does not entirely fill the mouth of the passage. The experiments of Michelotti and Castel determined a mean coefficient of discharge for cylindrical mouthpieces of 0·82, the extremes being 0·803 and 0·830; the singular effects produced under some circumstances by the application of cylindrical mouthpieces are more curious than useful. Conical converging mouthpieces increase the discharge more highly: the experiments on them of Castel, engineer of the waterworks of Toulouse, are exceedingly interesting; they demonstrated that under varied heads the coefficients of discharge and of velocity were practically constant for the same mouthpiece, and that for the same orifice of exit the coefficient of discharge increased from 0·83 for a cylindrical mouthpiece in proportion to the increase of the angle of convergence of the mouthpiece employed up to 0·95 for an angle of $13\frac{1}{2}°$; and that beyond this angle the coefficient of discharge diminishes to 0·93 for 20°, and afterwards decreases more rapidly. The length of mouthpiece employed in these cases as well as in the former was $2\frac{1}{2}$ times the diameter of the orifice. Some experiments by Lespinasse on the canal of Languedoc showed the enormous increase of discharge effected by using converging mouthpieces: his mouthpieces were truncated rectangular pyramids 9·59 feet long, the dimensions at one end 2·4 × 3·2 feet, at the other ·44 × ·62 feet, and were used in mills to throw the water on to water wheels; their opposite faces were inclined at angles of 11° 38′ and 15° 18′, and the head employed was 9·59 feet; the experiments resulted in deter-

mining a coefficient of discharge varying from 0·976 to 0·987.

Conical diverging and trumpet-shaped mouthpieces still further increase the discharge from an orifice: the experiments of Bernouilli, Venturi, and Eytelwein have thrown much light on this subject, and showed the coefficient to lie between 0·91 and 1·35. Venturi concluded that the mouthpiece of maximum discharge should have a length nine times the diameter of the smaller base, and a flare of 5° 6′, and that it would, if properly proportioned to the head of pressure, give a discharge 1·46 times the theoretic unreduced discharge through an orifice in a thin side.

Overfalls and Weirs.

An overfall may be considered to be a wide rectangular orifice in an ultimate position, where the head on the upper edge is zero; and its discharge may be therefore computed in the same manner as that of an orifice.

The discharge of an orifice is according to the parabolic theory—

$$Q = m \times \tfrac{2}{3} \sqrt{2g} \times w \left(H \sqrt{H} - h \sqrt{h}\right)$$

where h and H are the heads on the top and bottom edge, and d and w are the depth and width of the orifice; if then H = mean head on the centre of the orifice, and the orifice becomes an overfall, this formula becomes

$$Q = m \times \tfrac{2}{3} \sqrt{2g} \times w \left\{\left(H + \frac{d}{2}\right)^{\frac{3}{2}} - \left(H - \frac{d}{2}\right)^{\frac{3}{2}}\right\}$$

developing this, and putting wd = A, the sectional area,

$$Q = m\, A\, \tfrac{2}{3} \sqrt{2g\, H} \left(1 - \frac{d^2}{96\, h^2}\right)$$

and as d is comparatively small, the last term is = O, hence

$$Q = m\, A\, \tfrac{2}{3} \sqrt{2g\, H}; \text{ and } V = m\, \tfrac{2}{3} \sqrt{2g\, H}$$

where H is the head on the sill of the overfall.

The value of the coefficient, m, varies according to the form of overfall. It was determined by M. Castel, at Toulouse, by a large series of experiments: and also by Francis, in the Lowell experiments referred to in Chapter II., on Gauging.

The experiments of M. Castel showed that, for the accurate employment of a general coefficient of discharge, the dimensions and conditions of an overfall should fall within one of the three following classes.

1st. When the length of the overfall sill extends to the entire breadth of the channel, and the head on the sill is less than one-third the height of the dam or barrier, the coefficients remain remarkably constant, varying only from 0·664 to 0·666. Hence generally for this case, $m = 0·666$.

2nd. When the length of the overfall sill is less than the entire breadth of the channel of supply, but is greater than a quarter its breadth, the coefficient lies between the two extremes of 0·666 and 0·598, and is strictly dependent on the ratio of the length of sill to breadth of channel;—hence it is for the following relative breadths:

Relative breadth.	Coefficient.	Relative breadth.	Coefficient.
1·00	0·666	·50	0·613
·90	0·658	·40	0·609
·80	0·647	·30	0 600
·70	0·635	·25	0·598
·60	0·624		

3rd. If the length of the overfall sill is equal, or even only nearly equal, to one-third the breadth of the channel, the coefficient remains very constant, varying only between 0·59 and 0·61. Hence generally for this case, which is particularly favourable for gauging small streams, $m = 0·60$.

In other cases, that is, when the length of the sill is less than a quarter the breadth of the channel of supply,

the coefficient depends on the absolute length of sill, and requires determining specially: it increases from 0·61 to 0·67 in direct proportion to the diminution of absolute length of sill.

With reference to the three cases suitable for practical purposes, the experiments of M. Castel showed that when the sectional area of the overfall was less than one-fifth of that of the normal section of the channel of supply, the effect of velocity of approach in the channel did not modify the value of the coefficient: for other conditions, the modification necessary was not determined in a very satisfactory form:—the new equation for mean velocity of discharge being changed from

$$V = m\,\tfrac{2}{3}\sqrt{2g\,H}$$

into
$$V = m\,\tfrac{2}{3}\sqrt{2g\,(H + \cdot035\ W^2)}$$

where W = the surface velocity of approach, not determined from observation, but from its assumed ratio to the mean velocity, perhaps therefore the modification of the coefficient, m, by other authors into a new coefficient

$$m_1 = m\left\{\left(1 + \frac{h}{H}\right)^{\frac{3}{2}} - \left(\frac{h}{H}\right)^{\frac{3}{2}}\right\}$$

where h is the head due to the velocity of approach, and H is the head on the weir sill, is a preferable arrangement.

For the special cases in which channels are attached in continuation of the sides of the overfall, the coefficients in the experiments of Poncelet and Lesbros were reduced by 18 to 33 per cent. If, however, the fall to the channel is more than 3 feet, no reduction is generally made in the coefficients.

It may be noticed that the head on the sill used in the above expression is that in the centre of the overfall, which is independent of the rising of the water at the wings, a phenomenon to be observed in almost all cases of weir discharges.

In all the above cases, it is supposed that thin edges, as of metal sheets, or one-inch waste-boards, are used; for broad or round lopped crests, the coefficients will require reduction. See the coefficients given in Part 5, of Table XII., page lxxxiii.

Obstructed Overfalls.—When obstacles occur on the sill of an overfall, as dwarf pillars or blocks, a deduction in the discharge over the sill is made not only on account of the reduction of section, but on account of the contractions resulting. Francis's formula is applicable to these circumstances in cases where the length of weir sill equals or exceeds the head ;—it is

$$Q = \tfrac{2}{3} m \sqrt{2g} \cdot (l - 0{\cdot}1\, n\, H)\, H^{\frac{3}{2}}$$

where n = the number of end contractions,
= $2\cdot$ when there is no central obstruction,
l = length of weir sill,
$l H$ = A the sectional area of discharge,
and m = $0{\cdot}6228$.

In case the weir sill has the same breadth as the channel of supply, $n = 0$; and in that case

$$Q = 3{\cdot}332\, l H^{\frac{3}{2}}.$$

This, it will be observed, varies from that of Castel, which, under the same conditions, gives $Q = 3{\cdot}557\, l H^{\frac{3}{2}}$.

Partly Drowned Overfalls.—When a weir has its tail water above the edge of the sill, it may be treated as a combination of an overfall with an orifice; the upper portion down to the level of the lower water as an overfall, and the lower portion from that down to the sill level as a rectangular orifice, and the discharges calculated separately for each. Using, however, the same value of H in both cases, H being the head due to the overfall, that is, down to the level of the tail-race.

Some further values of coefficients of weir discharge are given in the accounts of gauging in Chapter II. To aid in the computation of discharges from overfalls, the velocities of discharge due to various heads and various coefficients may be obtained from those given in Table IX., pages xxxvii to xlv., by reducing the velocities there given by one-third; the results multiplied by the section of overfall are then the required discharges. The method thus adopted enables the same table to be used in computing the discharges of both orifices and overfalls. A table of weir coefficients is given on page lxxxiii., and some explanatory examples on pages xlvi. to xlviii.

11.—EFFLUX OR DISCHARGE FROM PRISMATIC VESSELS OR RESERVOIRS.

The following formulæ given by d'Aubuisson may be considered useful for reference in the cases in which they are required in engineering practice:—

First Case.

(1st.) When the reservoir empties itself through an orifice.

Velocities.—The ratio between the velocity at the orifice of discharge and that of the water in the reservoir is in the inverse ratio of their sectional areas.

Head.—If H = actual height of water in the reservoir; h = the height due to and generating the velocity of discharge, and A and a are the sectional areas of the reservoir and the orifice.

Then $h = \dfrac{\mathrm{H\,A}^2}{\mathrm{A}^2 - m^2 a^2}$.

Discharge.—A reservoir emptying itself through an orifice in a given time would discharge a volume equal to half that due to the head at the commencement, kept

constant during the same time. For an example of this applied to locks, see example 4, page xlvi.

Time.—The time in which a prismatic reservoir empties itself is double that in which the same volume would be discharged if the initial head had remained constant.

The time of descent, t, to a given depth, $d = H - h$

$$t = \frac{2\,A}{m a \sqrt{2g}} (\sqrt{H} - \sqrt{h}),$$

and the quantity discharged in a given time, t,

$$\text{is } Q = A\,(H - h) = \frac{t.m.a \sqrt{2g}}{A}\left(\sqrt{H} - \frac{tma\sqrt{2g}}{4A}\right)$$

and the mean hydraulic head, H, under which the same quantity would be discharged in the same time is—

$$H_1 = \left(\frac{\sqrt{H} + \sqrt{h}}{2}\right)^2$$

where H and h are the heads at the beginning and end of the time of discharge, the reservoir receiving no supply during that time.

(2nd.) When the basin or reservoir receives a constant supply during the time of discharge.

If q = quantity supplied per second,

t = time in which the surface will descend the depth, $x = H - h$.

$$t = \frac{2\,A}{(ma\sqrt{2g})^2}\left\{ma\sqrt{2g}(\sqrt{H} - \sqrt{h}) + q.hyplog\frac{ma\sqrt{2gH} - q}{ma\sqrt{2gh} - q}\right\}$$

when there is no supply, or $q = 0$, this equation resolves itself into that previously given.

(3rd.) In the case of there being no supply, but the discharge instead of being effected through an orifice is conducted over an overfall—

$$t = \frac{3\,A}{mL\sqrt{2g}}\left\{\frac{1}{\sqrt{h}} - \frac{1}{\sqrt{H}}\right\}$$

Non-prismatic reservoirs are extremely difficult to deal

with, and the investigation of any special case would be comparatively useless.

Second Case.

When one reservoir empties itself into another.

(1st.) When each of the two reservoirs being exceedingly large practically preserves its own level, the communicating sluice being below the lower surface of water; then if H, h, are the heads—

$$\text{the discharge } Q = ma\sqrt{2g(H - h)}.$$

(2nd.) When the upper reservoir being exceedingly large preserves its own level, and the lower reservoir having a definite area (A), receives the supply through a sluice of a section (d), required the time in which the surface of the lower basin will rise to a certain height.

If H, h, be the heads on the lower surface at the beginning and end of the time, t,

$$t = \frac{2A}{ma\sqrt{2g}}(\sqrt{H} - \sqrt{h}),$$

this formula, like that previously given, is useful for determining the time necessary to fill a lock chamber: when $h = 0$, or the levels become the same, the case is that of canal locks, and the sectional area of the sluice may be determined from this equation.

(3rd.) When neither reservoir receives any supply, and both are limited in size, if the surfaces are originally at different levels, and the communication sluice is opened, the surface of one will rise and the other fall.

If A, B, are the sections of the two vessels,

H, x, the heads at the beginning and end in A,

h, y, the heads at the beginning and end in B,

a = the sectional area of the pipe or sluice,

t = time during which the sluice is open,

$$\text{then } t = \frac{2A\sqrt{B}}{ma.(A+B)\sqrt{2g}}\left\{\sqrt{B(H-h)} - \sqrt{(A+B)x - AH - Bh}\right\}$$

and if it be required to know the time in which the two surfaces will be level; in that case, $x = y = \frac{AH + Bh}{A + B}$, and then

$$t = \frac{2AB\sqrt{H - h}}{ma(A + B)\sqrt{2g}}.$$

This formula is convenient for determining the time occupied in bringing the water in the two chambers of a double lock to the same level, by means of a sluice of known dimensions.

12.—THE APPLICATION OF THE WORKING TABLES.

The use of the greater portion of these twelve tables has already been indicated in the foregoing text; they have for their object not only the reduction of labour in calculating quantities, but also to serve as a check on any calculations of the same nature that may be rapidly made by engineers in dealing with quantities of water. Table I. gives the amount of the force of gravity in different latitudes, and may occasionally be found of use in pendulum experiments, and in such calculations in which the ordinary value of g 32·2 feet per second, generally applied in the hydraulic calculations in the form of $\sqrt{2g} = 8{\cdot}025$, is not sufficiently exact. Tables II., III., and IV., are of use in calculations of water supply from catchment areas, storage, flood discharge, and waterway. Table V. gives some velocities usual under certain circumstances that are occasionally required, and as to which the memory cannot always be trusted. Table VI. affords a ready means of reducing or converting gradients and angular slopes into the forms most usually required by hydraulic engineers. Table VII. gives mean velocities of discharge of open channels of all sorts; these have, however, in conformity with

modern practice, to be modified by coefficients suited to the particular case under consideration; the various functions of mean velocity can also be easily deduced by the aid of this table. Table VIII. gives discharges of full cylindrical pipes and tubes, and the diameters and heads corresponding to discharges; these also require modification by suitable coefficients in the same way. Table IX. gives velocities of discharge of sluices, the same table serving also for weirs by making a deduction of one-third from the velocities there given. Table X. gives the loss of head due to bends in open channels and in pipes, and the rise of water due to obstructions in open channels and rivers. Table XI. is a table of equivalents, affording the means for a ready conversion of quantities often entering into hydraulic calculation, such as total into continuous quantities; and, especially intended for use in calculations of storage, town supply, and distribution of water in irrigating land. The latter portion of this table consists of conversion tables for English and metrical measures. The greater portion of Table XII. is a collection of all the experimental coefficients necessary in ordinary hydraulic calculations; they have been arranged in this manner in preference to being distributed throughout the tables, in the belief that it permits of greater convenience in reference: part 6 of this table is a small collection of hydraulic memoranda, principally for purposes of conversion, and also of weight and pressure, intended to aid in rapid calculations; and part 7, consisting of useful numbers, having the same object, also serve for readily applying powers and roots to the coefficients that have now become so important a part of all hydraulic calculations. These tables and data have all been calculated and reduced by the author, with the exception of those at pages lviii and lx.

The Appendix to the Working Tables consists of a few miscellaneous tables and data, giving information sometimes required by the engineer in connection with hydraulic works, the last being a table of British-Indian weights and measures; these with two or three exceptions, in which the tables were made by the author, have been taken from the best sources available, and rearranged in a convenient form.

CHAPTER II.

On Field Operations and Gauging.

1.—DIRECT MEASUREMENT OF DISCHARGE.

The direct measurement of the discharge of a channel or stream can be obtained by means of gauge-wheels. The channel is widened until the water flows at a moderate depth, less than five feet, over a horizontal and carefully constructed apron which is divided by piers into a number of equal openings. At each of these openings a gauge-wheel is placed, which fits the opening every way within a quarter of an inch. Sheet piling is driven across the head of the apron and along the banks approaching it for some little distance, so as to force the whole of the water of the stream to pass between the piers and drive the wheels. The measurement of the water is

determined by the number of revolutions of the wheels, which should be all coupled on to one shaft and be made self-recording on a dial-face, and by the dimensions of the wheels, or spaces between their blades, as well as by the depth of water passing over the apron, which is observed at intervals of about five minutes on gauges erected for the purpose.

The method of obtaining a discharge by means of gauge-wheels is expensive and interferes with navigation as well as the passage of the water; it is therefore very rarely adopted.

2.—GAUGING BY RECTANGULAR OVERFALLS.

The water of a canal or stream is made to discharge itself over a single horizontal dam, or over a series of small overfalls specially constructed for the purpose. The discharge over overfalls of certain dimensions, and under certain circumstances, is known by many series of experiments to be correctly expressed by a formula, containing the required data and dimensions, known as Francis's formula, it is

$$Q = \frac{2}{3}\sqrt{2g} \times m \quad \Big[l - 0{\cdot}1\, n\, H\Big] H^{\frac{3}{2}}$$

where l = length of weir-sill.

H = head on the weir from still water.

n = number of end contractions.

If the weir-sill is of the same length as the breadth of the channel of approach, $n = 0$; if less than it, and there is no central pier or obstacle, $n = 2$; each central obstacle involving two additional end contractions.

taking $\sqrt{2g} = 8{\cdot}025$ and $m = {\cdot}6228$

$$Q = 3{\cdot}33198 \Big[l - 0{\cdot}1\, n\, H\Big] H^{\frac{3}{2}}$$

This gives results within one per cent. of absolute

exactitude. The dimensions in this formula being taken in feet, the discharges will be in cubic feet per second.

The following conditions should be observed in gauging by rectangular overfalls.

1. As regards form of construction, the dam in which the overfall or series of overfalls is placed, should have the sills truly horizontal, and the sides of the overfalls truly vertical: the dam itself should be vertical all along on the up-stream side, but the sills should all be sloped off on the down stream side at an angle of 45° or more with the horizon; all the edges of discharge should be sharp and true, after passing which the water should discharge itself unobstructed.

2. In order to obviate the necessity of allowing for the velocity of approach in the channel, the area of the overfall—*i.e.*, the quantity $l \times H$, must not exceed one-fifth the area of the channel; otherwise an allowance must be made on this account, as given in the paragraph on Weirs, Chapter I.

3. If the velocity of the channel of supply should not be uniform in all parts of its section, arrangements should be made to make it so; this can be done by placing gratings, having unequally distributed apertures, all across the channel, and as far from the overfall as possible, and letting the water pass through them under a small head.

4. In addition to the above it is absolutely necessary that the air under the falling sheet of water should have free communication with the external air.

With regard to dimensions:—

5. Should the overfall not extend to the entire width of the channel of supply, there should be at least a difference at each end equal to the depth on the overfall, so as to produce complete end contraction.

6. When the breadth of the overfall is equal to that of

the stream, and even under all circumstances, the depth on the weir should be less than one-third the height of the barrier.

7. The depth on the weir must be always less than one-third of the length of the sill.

8. The head on the overfall, H, should never be less than ·2 feet; it is better, also, to make it more than ·5 feet and less than 2 feet.

9. The fall from sill to tail-water should not be less than half the depth on the weir in order to ensure a free fall.

The following practical directions suitable to streams and moderate rivers are given as examples, where ordinary care and accuracy is required.

Practical directions.—1st. When the discharge is supposed to be less than 40 cubic feet per second:—

First, according to the above rules, make H greater than ·2 feet; and $H \times l$ less than one-fifth of the channel section; let l be greater than ·3 feet, but less than one-third the width of the channel; and, to ensure a free fall, arrange so that the lower edge of the sill may not be less than half a foot above the tail-race.

Under these conditions the coefficient of discharge to be used will be $m = \cdot 623$, and any error should not be more than one per cent. Obtain the surface velocity (V_s) and the transverse section (S): the approximate discharge will then be $Q_s = V_s \times S$, and assuming a value for l as before mentioned, obtain a value for H by means of the ordinary formula, making use of the approximate discharge for this purpose. H should be from 1 to 3 feet, and should such a value not result, from the application of the previous conditions, use another value for l, so as to secure this condition, as well as to retain the other conditions before mentioned. When this is gained, the orifice may be cut of the required dimensions in one-inch plank well puddled,

of which such dams are usually made; and as, in practice, the dimensions are not likely to be very closely adhered to, they should be measured again when the orifice is completed, and applied to the formula before given for this purpose to obtain the velocity of discharge and amount of discharge.

2nd. When the supposed discharge is more than 40 cubic feet per second, but still admits of being dammed:—

Find the approximate discharge from the section and velocity, when the surface of the stream is level with a fixed mark on a post or stone, at from 100 to 200 feet below the intended site of the weir; having previously selected a place where the stream is regular in width and inclination, construct the dam so that the weir-sill may be equal to the full breadth of the channel, square the ends of the opening with planking, and put a gauge at each end, with the zero at the level of the upper edge of the sill of the overfall, which again should be from 1 to 5 feet above the fixed bench-mark.

When the water is up to the mark, read the height on either scale; take their mean, and use it as a value for H in the weir formula before given to obtain the velocity and amount of discharge. If necessary, obtain the surface velocity of approach W, and make allowance for it as before mentioned under the head of weir discharges, as suitable for this case; *m* being = 666.

3.—THE MEASUREMENT OF VELOCITIES.

There are many cases when it is not advisable to construct a dam or gauge by overfalls, and also cases where the simple calculation of discharge due to the slope of the river, and the terms of its cross section, would not give *sufficiently* accurate results. Under these circumstances

velocity observations must be made, and other data correctly obtained, so as to obtain from them the mean velocity of discharge, which, when multiplied by the sectional area, gives the required discharge.

In all cases where velocity must be observed, it is necessary to choose a straight reach of the river having a tolerably uniform channel section; it is also advantageous that the bank should admit of the measurement of a straight line parallel to the general direction of the channel, and at right angles to the line of intended river section of observation, to serve as a base for triangulation.

For exactitude of result, it is also advantageous where circumstances admit of it to use a flume, should the channel be sufficiently small to admit of it, as this ensures a perfectly regular section of water for a certain distance, and admits of more exactitude in the determination of the sectional area and that of the hydraulic mean radius. A flume is a timber framework covered with carefully jointed plank, forming a complete lining to the bottom and sides of the channel for from 100 to 200 feet in length, having a perfectly equal section throughout; this gives the means of accurately measuring the dimensions of the stream, the whole of the water of which is forced to pass through it by means of sheet piling at its upper entrance. It produces no sensible disturbance in the flow of the water, and does not interfere with the navigation or passage of water. Velocity observations are then made on a measured length along the flume to obtain the mean velocity, which, when multiplied by the section of the flume, gives the required discharge. A long and accurately constructed open aqueduct in perfect order answers all the purposes of a flume. Should, however, no such opportunities for the exact determination of the water section present themselves, it becomes necessary to resort to soundings. These are perhaps

best and certainly most rapidly taken by means of a surveyor's 100 feet chain, with a suitably heavy leaden weight attached to one of the handles; some, however, prefer a cord to a marked chain, and consider it better to measure the length of cord with a tape at each sounding.

The determination of the position of each sounding can in narrow reaches of rivers be best made by stretching a rope across the river, and measuring the distances of the sounding points from one bank along the cord. In wide reaches where this is impracticable, the sounding points have to be fixed by angular observation and connected with the base line of triangulation at the moment of sounding either by an observer with a theodolite on the shore, or by one in the boat with a pocket sextant.

The fall of the water surface at all states of the river is one of the data generally required. To determine this, a gauge post is erected, driven into the ground at each sounding section, and the heights of the water shown on them continually recorded so as to show all variations of depth; the connection of level between the two or more gauge posts is made by levelling either from one post to the other, or from both to a fixed bench-mark. In many cases the fall of the water surface is so slight that the ordinary 14-inch level, and staves graduated to hundredths of a foot, of the ordinary surveyor, do not give sufficiently exact results, when a good 18-inch level and staves reading to millimetres might perhaps just answer all purposes.

The gauging of the exact water level, the variations of which are frequently very small though still important, often requires arrangements giving greater precision than that given by a gauge post, or a rod held to the water level. The two instruments employed for arriving at a very exact determination of water level are—1st, Boyden's hook gauge; 2nd, The tube gauge, used by Bazin.

Boyden's hook gauge.—With regard to gauges, it is well known that the capillary attraction of water about any rod placed in it as a gauge for determining the water level will falsify readings; to obviate this the well-known Boyden's hook gauge may be used where extreme precision is necessary. This gauge has a hook at its lower end, which can be raised or lowered by turning a screw; when the point of the hook is even a thousandth part of a foot above the water surface, the water around it is sensibly elevated by the capillary attraction, and obviously distorts the reflection of light from the surface; when the hook is lowered just sufficiently to cause this distortion to disappear, the point of the hook must coincide with the water surface; a true reading, exact within ·001 of a foot, can then be read, by means of a vernier attached to the rod of this gauge which is graduated to hundredths of a foot. As this instrument cannot be used effectively in a current, it is usual to put it in a box in some convenient place which only communicates with the external water by means of a hole, or if the depth at some distance off is the object, by a pipe leading from that place to the hole in the box; any oscillation of the water surface in the box may then be diminished or nearly removed by partially obstructing the hole at will. Should perfect rest not be attainable, a good mean position of the point of the hook may be obtained by adjusting it to a height at which it will be visible above the water surface for half the time. It is sometimes convenient to have the hook made with a small semispherical knob on it, a level-staff can then be held on it for taking a sight with an instrument.

The tube-gauge used by Bazin is, unfortunately, not described in detail, nor are drawings of it given in his "Recherches Hydrauliques." It seems, however, to have been a glass tube having a mouthpiece of only a millimetre

in diameter, and that it enabled variations of water level of one millimetre to be easily read; and it is hence extremely probable that it resembled in some respects the velocity gauge-tube of d'Arcy, used for taking velocity measurements, hereafter described. It is, in fact, evident that an instrument on this latter principle, capable of indicating variations of velocity with precision, would also indicate with exactness the moment of the withdrawal from, or submersion of its mouthpiece in, the water, and that this motion could be easily manipulated with a clamping and a tangent screw.

In addition to the above data, it is also advisable to take notes of the nature and quality of the soil of which the bed and banks of the river under consideration are composed, as these have an important effect on the discharge, and to notice what amount of velocity of current is just sufficient to cause erosion in them.

The different modes of measuring velocity are the following:—

Surface velocity is very simply measured by observing the time of transit over a known distance or length of a reach of a river, of any light floating body, a wafer, a ball of wood or cork, or a partly filled bottle.

Mean vertical velocity, or the mean of all the velocities from water surface to the bottom under any point, in a vertical plane, is measured by a rod placed vertically, having a length nearly equal to the depth of the river, loaded at one end, and supported by a float at the upper end. The time of transit of such a rod will then give approximately the mean velocity of the vertical plane in which it moves. These rods or poles are sometimes made hollow and weighted inside, as the painted metal tubes of the Lowell experiments hereafter mentioned, thus obviating the necessity of attaching either floats or weights.

A more convenient mode of observing mean vertical velocity consists in lowering from the surface to the bottom, and raising again to the surface any accumulative self-recording current meter. This is an operation requiring extreme care; the meter must be sufficiently weighted, and, if necessary, also managed by a cord from an additional boat moored up stream so as to ensure its moving vertically up and down; the lowering and raising of the meter must also be evenly and steadily managed, so that the results may not be falsified.

Mean sectional velocity can be approximately obtained in small streams and canals at one operation only by making a light covered framework nearly the size of the whole cross-section of the stream, and so arranging it by floats and weights that it will assume a vertical position at right angles to the thread of the current; its time of transit can then be noted, and this will be the approximate mean velocity of the section.

Sub-surface velocities.—The following are means and appliances for measuring the force of a current, but most of these involve the application of a special coefficient of reduction due to the particular appliance, in order to obtain the actual velocity in feet per second at any depth:—

1.—By double floats.

A weighted float, consisting of ball, or cube of wood, or hollow tin weighted with lead, is sunk to the required depth, being attached by a cord to a small upper float on the surface of the water; the upper float being made of cork, light wood, or hollow tin, carrying a vertical stick, or wire, for convenience of observation, and the length of cord being so adjusted as to prevent the weighted float from sinking lower than the depth at which the current velocity is required. The time of transit of this double float, over a

measured or a calculated distance, is observed, and is supposed to represent the velocity of the stream at that depth, independently of any coefficient of reduction.

Another method is to employ a pair of equal hollow balls connected or linked together, the upper one on the surface, and the lower one weighted sufficiently to keep it at the certain depth; the velocity of this double float, as observed on a measured distance, is supposed to be that of the current at half the depth of the lower ball.

2.—By instruments of angular measurement.

A quadrant having a graduated arc has a string attached to its centre, and a ball attached to the string, which is immersed in the stream. The current moving the ball produces an angular change from verticality in the position of the string; the velocity is then equal to the square root of the tangent of this angle multiplied by a coefficient, which is constant for the same ball only.

3.—By the indications of a balance.

A ball is immersed in the stream and attached by a wire to a balance, which registers the pressure. Another very similar method requires a small plate instead of a ball, which is connected with the balance, and which is directly opposed to the current.

The tachometer of Brünings is the best known instrument of this type. It consists of a plate fixed at one end of a horizontal stem, which moves in the socket of a vertical bar, by means of which the instrument either rests on the bottom of the channel or is suspended from above. A cord of fixed length is fastened to the other end of the stem, and, passing under a pulley, is attached to the short arm of a balance, on whose other arm a weight is suspended, being placed in such a position that the equilibrium is established with regard to the force of the current *under observation*. The position of the weight on the

graduated arm of the balance indicates the velocity observed.

4.—By the rotation of a screw.

A light metal screw, similar to that of a ship's patent log, will, when submerged in a current, rotate at a velocity approximate to that of the water in which it is placed. If on the axle of the screw a thread is set turning one or more worm-wheels, the number of revolutions of the worm-wheel will indicate the approximate velocity of the water, from which, by applying a coefficient of reduction applicable to the particular instrument, thus including all allowances for friction and other causes, the true velocity of the current may be obtained. There are several current meters of this type: Saxton's, Brewster's, and Révy's, hereafter described, are all modifications of this form. Some of these instruments are not suited to great depths and high velocities; others are made self-recording in such a way as to make allowance in the indicated number of revolutions for the loss of velocity by friction; the latter is a great disadvantage, as it is always practically necessary to test each particular instrument, and make use of a coefficient, however small it may be, in order to obtain accurate results.

The earliest now known instrument of this type is the hydrometric mill of Woltmann, used by him in 1790. The wings on its axle resembled those of a windmill, and were square copper plates, set at an angle of 45°, having their sides ·082 feet and their centres at ·164 feet from the axis of rotation; for small velocities the size and distance of the wings was doubled. In great depths this instrument was attached to a bar and lowered from a platform between two boats, and the instrument put in gear or out of gear by means of a cord at any depth. This type of current meter, from its convenience of use in observing velocity at any depth, has been re-invented many times.

5.—Pitot's Tube.

This is a glass tube bent at the lower end; it is sunk to the required depth, and its lower orifice directed against the current: the velocity is deduced from the difference of level between the water in the river, and that in the tube which is forced up by the current. The first improvement of this instrument is that of Dubuat, who gave the orifice of the tube a funnel shape, and closed it by a plate pierced with a small hole, thus considerably reducing the objectionable oscillations of the water in the tube. The next is by Mallet, who terminated the horizontal branch of the tube by a cone, having an opening of 2 millimetres, and made the tube itself of iron with a diameter of 4 centimetres; he also introduced a float and stem which, elevated by the force of the current, indicated heights on a graduated scale. The last improvement was that of d'Arcy, hereafter described.

6.—Grandi's Box.

A box, having a small hole in the side towards the current, is sunk to a certain depth and withdrawn after a certain time; the amount of water in the box indicates the velocity at that depth.

7.—Boileau's Air Float.

A glass tube of fixed length is immersed in a position parallel to the current; the upper end of the tube has a conical mouthpiece fitted to it of any convenient size; the velocity of passage of a globule of air through the tube indicates the velocity of the current.

Some of these modes of measuring velocity have for the present practically fallen into disuse, on account of the very limited range of their applicability; others, on the contrary, have been severally adopted by various hydraulicians in modern times, to the entire exclusion of the rest.

Modes adopted in Modern Practice.

(1.) On the Mississippi Surveys it was determined to use the most simple apparatus, so as to avoid the necessity of applying any coefficients of reduction to the velocities indicated by them; and double floats were invariably used. The floats used in the Mississippi Survey were kegs without top or bottom, ballasted with strips of lead, so as to sink and remain upright; they were 9 inches in height, and 6 inches in diameter; the surface floats, when of light pine, $5{\cdot}5 \times 5{\cdot}5 \times {\cdot}5$ inches, when of tin, ellipsoids, axes 5·5 and 1·5 inches, the cord one-tenth of an inch in diameter; for observations more than 5 feet below the surface, the kegs were 12 inches high by 8 inches in diameter, and the cord nearly two-tenths of an inch; neither the weight of the surface float nor the force of the wind directly affected the observations to any appreciable amount.

(2.) On the gauging of the Paranà and La Plata, by Mr. Révy, the screw current meter, with some alterations and improvements made by him, was invariably adopted.

For ordinary currents the screw used by Mr. Révy consisted of two long thin blades of German silver, having a diameter of 6 inches, and a pitch of 9 inches; the thread of its axis worked on two worm-wheels of 3 inches in diameter, one wheel having 200, and the other 201 teeth; each revolution of the screw moved the first wheel one tooth onwards, the second wheel moving one tooth onwards for each complete revolution of the first wheel; this allowed of the continuous reading of 40,000 revolutions; the two worm-wheels had graduated divisions around their circumferences, corresponding to the teeth in number and position, which were read off at an index through a glass plate covering them. A nut was also used

for clearing the worm-wheels from the thread of the axle of the screw, by means of which the instrument was either put in gear or out of gear by hand; a wire attached also enabled this to be done from above when the instrument was at any depth.

For strong currents, the screw-blades were shorter and stronger, and made of steel. Some of the screws used were only 4 inches in diameter. The divisions on the circumferences of the wheels were found to be too near for convenient reading; 100 and 101 divisions would have been preferred to the existing arrangement of 200 and 201.

These meters were generally used for observing velocities of more than 10 feet per minute, their corrected results being absolutely correct within 1 inch per minute of velocity. They required extreme care and continual watching: the slightest bend or damage to a screw-blade, or any clogging or accidental tightening of a screw being liable to vitiate results.

When in good order, exposure to a gentle breeze is sufficient to keep the instrument revolving;—failing this, cleaning and oiling, or readjusting carefully, is absolutely necessary. In order to keep a check on the observations, a second current meter should always be at hand.

The principal advantage of this description of current meter is the convenience with which it can be worked, and its unvarying utility in observations at any depth of water.

(3.) In the experiments of d'Arcy and Bazin, on the Rigoles of Chazilly and Grosbois, the gauge-tube of d'Arcy, a development of the tube of Pitot, was generally used for taking velocity observations.

Pitot's tube, used in 1732, demonstrated the principle that the difference of water level, *h*, shown by the two tubes, one vertical and the other curved, and directed *against* the current, was that due to the velocity, and that

the latter could be obtained from the former, by making use of the formula $V^2 = 2gh$.

The error in this was caused by the fact that the water in a vertical tube immersed in a current stands lower than the water surface outside; the difference being a quantity dependent on the square of the velocity immediately below the orifice. In addition to this Pitot's tubes had a serious disadvantage in that the oscillation of the water within the tubes, whose orifices were of the same diameter as the tubes themselves, did not allow the difference of level to be correctly observed.

These objections are entirely removed in the improved tube of d'Arcy, which has an orifice 1·5 millimetres in diameter for a tube one centimetre in diameter: in addition to this the lower portions of the tube to which the orifices are attached, have a small diameter, and are made of copper: besides this, two cocks are introduced which add greatly to convenience of manipulation. The lower cock, which can be worked by a wire and lever, enables the orifices to be opened or closed at any moment from above, and thus allows the difference of water levels of the tubes to be read off at leisure, after withdrawing the instrument from the water. The upper cock, after the water in the tubes is drawn up by the breath at an upper orifice, shuts off the air, and enables the difference of water level in the tubes, which is not affected by dilatation or compression of the atmosphere, to be read off above against a scale.

This gauge-tube is described in "Les fontaines publiques de la ville de Dijon, 1856," and drawings of it are given in the "Recherches Hydrauliques" of d'Arcy and Bazin, 1865.

In the latter, the vertical glass tubes are 1·25 m. long, the two small copper tubes below them being enclosed in a copper casing, 0·77 m. long, 0·06 m. broad, and 0·011 m.

thick, terminating in a sharp wedge-shaped point to reduce the effect of the perturbation of the current. The tubes themselves are affixed to an upright of light boxwood, which is graduated and supplied with a vernier; the whole instrument being attached to an iron standard on which it slides, and to which it can be fixed by screws at any height; a handle turning the instrument directs the orifices in any required direction; and an additional movable wooden arm is used to enable the instrument to rest by means of it on any cross-beam or timber from which the observations are being taken.

In taking an observation with the instrument it is usual to take a mean of three maxima and minima.

The following is the theory of the determination of the coefficient of reduction μ in the formula $V = \mu \sqrt{2gh}$ for any instrument.

If a single curved Pitot tube be placed in a current, first, with its orifice directed against it, and recording a height, h', above the natural water surface; secondly, when directed with it, and recording a loss of level, h'', below that of the natural water surface; and thirdly, when directed at right angles to the current, recording a loss of level h''', then—

$$\frac{V^2}{2g} = m'h' \,; \quad \frac{V^2}{2g} = m''h'' \,; \quad \frac{V^2}{2g} = m'''h''';$$

and hence—

$$V = \sqrt{\frac{m'm''}{m' + m''}} \sqrt{2g\,(h' + h'')} = \mu \sqrt{2g\,(h' + h'')}$$

$$V = \sqrt{\frac{m'm'''}{m' + m'''}} \sqrt{2g\,(h' + h''')} = \mu' \sqrt{2g\,(h' + h''')}$$

and finding from tables the values of velocities V' and V'' corresponding to the heights $h' + h''$ and $h' + h'''$; the above equations become—

$$V = \mu\, V' \,; \text{ and } V = \mu'\, V'';$$

hence there is a constant relation between the theoretic height $\frac{V^2}{g}$ due to the velocity of the fillet under consideration and the quantities h', h'', h'''; and the coefficient of reduction can therefore be obtained for any sort or form of orifice by means of a few experiments; also, when once the coefficient of reduction for the instrument is determined, it is unnecessary to make further use of the level of the water, in which the instrument is plunged, in determining velocities.

4.—GAUGING CHANNELS BY MEANS OF SURFACE VELOCITIES ONLY.

The experiments of Messrs. Baldwin and Whistler on discharges of canals of rectangular section are worthy of notice. They obtained discharges on the canals by means of surface velocities and flume measurement, and simultaneously gauged the actual discharges by gauge wheels, with the view of determining practically the relation between surface velocity and mean velocity, for channels of a certain size conveying water at certain velocities.

In one case the flume was 27·22 feet wide, with depths of water from 7·52 to 8·14 feet, having surface velocities from 3·07 to 3·34 feet per second: the observations deduced a mean coefficient of velocity ·857, the extremes being ·838 and ·856.

In the other case, the flume was 29·94 feet wide, with depths of water from 7·67 to 8·85 feet, having surface velocities from 1·91 to 2·77 feet per second; the observations deduced a mean coefficient for the surface velocity of ·814, the extremes being ·797 and ·846.

In other cases, the data of which are not forthcoming, the coefficients of surface velocity were ·835, ·830, ·810; and taking ·829 as the mean of the five results, it can be

favourably compared with De Prony's coefficient ·816, obtained from experiments on wooden troughs 18 inches wide, having depths of water from 2 to 10 inches, and velocities varying from 5· to 4·25 feet per second. Another point which Messrs. Baldwin and De Prony agreed in determining was that their coefficients should be slightly reduced for lower velocities and increased for higher. The result is that the proportion between the surface velocity and the mean velocity of discharge for rectangular channels in plank, and within certain limits of velocity and proportions of cross section, may be said for practical purposes to lie between ·8 and ·85. Under similar local conditions, therefore, the discharge of a canal of rectangular section can be rapidly obtained by a few surface velocity observations, the inclination of the water surface, and the measurement of its section. The more recent experiments, however, of d'Arcy and Bazin show that the above law of velocity does not hold generally; and hence this mode of gauging does not admit of general application.

5. GAUGING CANALS WITH LOADED TUBES; BY FRANCIS.

Under the existing arrangements at Lowell, a daily account is usually kept of the excess of water, if any, drawn by each manufacturing company over and above the quantity it is entitled to under its lease. In ordinary times, occasional measurements are sufficiently exact; but when water is deficient, frequent measurements are made. In the latter case, the following is the usual course of proceeding:—

A gauging party, consisting of one or more engineers with assistants, is assigned to each flume where measurement is necessary; and arrangements are so made that the *observations for a* single gauging occupy about an hour,

the intervals during the day being occupied in working out the results, which are immediately communicated to the manufacturers, so that the machinery may be adjusted to the amount of water they are entitled to draw.

The following are the dimensions of the measuring flumes used, and the quantities of water usually gauged in them; the depth of water in the flume generally varying from 6 to 10 feet.

Merrimac	100′ long by	50′ wide,	1500	cub. ft. per sec.
Appleton	150	50	1800	do.
Lowell, M. C.	150	30	500	do.
Middlesex	150	20	200	do.
Prescott	180	66	2000	do.
Boott	100	42	800	do.

The loaded tubes used were cylinders 2 inches in diameter made of tinned plates soldered together, with a piece of lead of the same diameter soldered to the lower end, having sufficient weight to sink the tube nearly to the required depth, thus leaving generally about 4 inches above the water surface. A red-paint mark was made to show the amount of immersion required, leaving a space between the bottom of the tube and the bottom of the canal of 1 foot. The tubes were of thirty-three different lengths, varying from 6 to 10 feet: six of each length were provided for this purpose.

In order to adjust the tube precisely, it was placed in a tank made for the purpose, and small pieces of lead were dropped into the top of the tube, and rested on the mass of soldered lead, and more were added until the tube was sunk to the required depth, when the orifice at the top was closed by a cork. The tubes were allowed to remain floating for some time in the tank in order to discover any leak. If they leaked, they were taken out and filled

with water to discover the position of the leak, when the leak was soldered and the tube adjusted again. The centres of gravity of the tubes adjusted were 1·78 to 1·90 feet from their bottom ends; and thus being low, the tubes had a strong tendency to remain vertical.

The tubes were put into the water by an assistant standing on a bridge below the upper end of the flume, a thing requiring a little practice to do well; he stood with his face up-stream, with the tube in hand, the loaded end directed downwards, but slightly up-stream, holding it at an angle with the horizon, greater or less, depending upon the velocity of the current. At a signal he pushed the tube rapidly into the water at the angle at which he previously held it, until the painted work near the upper end of the tube reached the surface of the water; he retained his hold of the upper end of the tube until the current brought it to a vertical position, when he abandoned it to the current.

There were three transit timbers placed across the flume, the middle one equidistant from the other two, their up-stream edges vertical, and distinctly graduated in feet from left to right. An assistant stood at each transit timber to note the transits, the assistant at the middle transit timber also observing the depth of water in the flume at each transit in a box close to him between the lining planks and the wall of the canal, which communicated with the flume by a pipe about 4 feet above the bottom. The box contained a graduated scale, divided to hundredths of a foot, the zero point being at the mean elevation of the bottom part of the flume between the upper and lower transit timbers. The bottom of the flume was very nearly horizontal; the elevations to obtain the mean were taken at 32 points, giving an extreme *difference observed* of ·027 feet in one case. The course

of the tube, denoted by the distance in feet from the left side of the flume when the tube passes the transit timbers, was also observed and called out by the assistants; the mean course being obtained by adding the distances at the upper and lower transit timbers to twice that at the middle, and dividing the result by four for a mean distance.

The usual method of observing the transits was by means of an assistant carrying a stop watch beating quarter seconds, who walked down and recorded every transit himself; but when greater exactness was required, an electric telegraph made for the purpose was used, by which the transit observers communicated transits to a seated observer from their stations, the times of signals being noted by him to tenths of seconds, according to a marine chronometer placed before him beating half seconds:—an assistant was also required to carry back the tubes to the up-stream station. In the usual method before stated, a party of five was sufficient for all purposes. The observations were made at distances apart about 1·5 feet in the cross section, as may be seen in the following gauge record for one set of observations, and the mean velocities of the tubes for these mean distances calculated and plotted on a diagram of section paper having the mean widths in feet of the flume scaled on one side, and the other calculated velocities for those widths scaled on the other; a curve joining these points was then drawn on the diagram, from which the mean velocity for each foot in width of the flume was scaled off and entered in the record; from these the mean velocity due to the total width was obtained 2·4311 feet per second; and since the mean section of waterway between the upper and lower transit timbers was = 41·76 × 8·5294 = 356·155 square feet, the approximate discharge = 2·4311 × 356·155 = 865·929 cubic feet per second.

Gauge record of the quantity of water passing the Boott measuring flume, May 17, 1860, *between* 10.30 *and* 11.30 A.M., *length between transit timbers,* 70 *feet, breadth of flume* 41·76 *feet, length of immersed part of tube* 8·4 *feet.*

Position of tube at starting.	Mean velocity of transit.	Position of tube at upper transit.	Position of tube at lower transit.	Mean position.	Depth of water in flume.
0·0	2·102	·3	·8	·55	8·510
1·5	2·258	1·8	1·6	1·70	8·481
3·	2·318	3·2	2·1	2·65	8·450
4·5	2·473	4·4	4·5	4·45	8·470
6·	2·373	6·2	5·4	5·80	8·445
7·5	2·593	8·2	10·1	9·15	8·438
9·	2·672	9·7	10·4	10·05	8·440
10·5	2·800	10·5	8·8	9·65	8·470
12·	2·713	12·3	10·9	11·60	8·483
13·5	2·778	13·8	15·5	14·65	8·490
15·	2·800	15·2	18·0	16·60	8·500
6·5	2·373	17·0	20·4	18·70	8·498
18·	2·593	18·0	17·8	17·90	8·505
19·5	2·431	19·7	19·0	19·35	8·505
121·	2·280	21·1	20·9	21·00	8·522
22·5	2·201	23·4	29·3	26·35	8·533
24·	2·077	23·7	22·1	22·90	8·510
25·5	2·071	26·5	29·7	28·10	8·495
27·	2·258	27·0	25·2	26·10	8·483
28·5	2·258	28·6	26·5	27·55	8·495
30·	2·414	31·0	34·3	32·65	8·550
31·5	2·500	32·1	30·	31·05	8·630
33·	2·258	32·5	28·1	30·30	8·610
34·5	2·672	34·6	36·7	35·65	8·625
36·	2·431	36 5	35·0	35·75	8·632
37·5	2·456	37·5	35·5	36·50	8·612
39·	2·500	40·1	40·5	40·30	8·578
40·	2·500	39·0	39·6	39·30	8·578
41·	2·397	41·2	40·6	40·90	8·560
41·76	...	...	...	...	...
0·0	2·047	·5	·4	·45	8·471
10·	2·642	9·8	8·7	9·25	8·580
20·	2·174	20·9	19·9	20·40	8·605
30·	2·273	31·5	33·8	32·65	8·635
41·	2·295	41·4	40·6	41·00	8·610
41·76	...	...	...	...	...
				Mean	8·5294

Products of mean velocity and widths.

2·073 × 1	= 2·073
2·193 × 1	= 2·193
&c.	2·284
	2·359
	2·422
	2·478
	2·529
	2·577
	2·623
	2·666
	2·705
	2·744
	2·776
	2·801
Products of each foot of breadth.	2·811
	2·798
	2·747
	2·648
	2·514
	2·363
	2·242
	2·174
	2·129
	2·090
	2·108
	2·135
	2·160
	2·023
	2·243
	2·286
	2·339
	2 371
	2·413
	2·453
	2·483
	2·513
	2·530
	2·541
&c.	2·544
2·504 × 1	= 2·500
2·417 × 1	= 2·417
2·264 × ·76	= 1·721
Sum	101·523
Mean $\frac{101·523}{41·76}$	= 2·4311

To obtain the true discharge from this approximate result, an empirical factor, depending on the difference (D) between the depth of water in the flume, and the depth to which the tube was immersed, divided by the depth of water in the flume, was applied: the expression of correction being $1-0{\cdot}116\,(\sqrt{D}-0{\cdot}1)$. The value of this expression for various values of D is given in the attached table at p. 98.

In this case D, the quantity before mentioned,

$$=\frac{8{\cdot}5294-8{\cdot}4000}{8{\cdot}5294}=\cdot0152\,;$$

and hence the true discharge

$$=865{\cdot}929\times\left\{1-\cdot116\,(\sqrt{\cdot0152}-\cdot1)\right\}=863{\cdot}59.$$

Remarks on the application of this method of gauging.

The preceding measurements were made in a flume placed below a quarter bend in the canal, which caused the velocity to be much greater on one side than the other. To obviate this, an oblique obstruction was placed near the lower end of the bend, which removed all the trouble in measurement due to the original irregularity; the other remaining irregularities may be seen by plotting a diagram of the velocities. It is hence advisable in all cases to equalize the velocities on each side of the axis, should they require it.

In gauging a branch canal it is best to put the flume in it near its off-take from the main canal, with its axis nearly parallel to that of the branch canal. Its section may be determined by roughly calculating the expected discharge, and making it so as to suit a velocity of from 1 to 3 feet per second; its length should not be less than 50 feet, allowing 20 feet above the upper transit timber to enable tubes to attain the same velocity as the water, and 5 feet below the lower timber, the transit course of 25 feet, run

Table of correction for Discharges obtained from Tube Velocity observations, being values of the expression $1 - 0{\cdot}116\,(\sqrt{D} - 0{\cdot}1)$ *for different Values of* D *(from the Lowell Experiments).*

D	Correction.	D	Correction.	D	Correction.	D	Correction.	D	Correction.
·000	1·01160	·020	·99520	·040	·98840	·060	·98319	·080	·97879
·001	1·00793	·021	·99479	·041	·98811	·061	·98295	·081	·97859
002	1·00641	·022	·99439	·042	·98783	·062	·98272	·082	·97838
·003	1·00525	·023	·99401	·043	·98755	·063	·98248	·083	·97818
·004	1·04426	·024	·99363	·044	·98727	·064	·98225	·084	·97798
·005	1·00340	·025	·99326	·045	·98699	·065	·98203	·085	·97778
·006	1·00261	·026	·99290	·046	·98672	·066	·98180	·086	·97758
·007	1·00189	·027	·99254	·047	·98645	·067	·98157	·087	·97738
·008	1·00122	·028	·99219	·048	·98619	·068	·98135	·088	·97719
·009	1·00060	·029	·99185	·049	·98592	·069	·98113	·089	·97699
·010	1·00000	·030	·99151	·050	·98566	·070	·98091	·090	·97680
·011	·99943	·031	·99118	·051	·98540	·071	·98069	·091	·97661
·012	·99889	·032	·99085	·052	·98515	·072	·98047	·092	·97641
·013	·99837	·033	·99053	·053	·98489	·073	·98026	·093	·97622
·014	·99787	·034	·99021	·054	·98464	·074	·98004	·094	·97604
·015	·99739	·035	·98990	·055	·98440	·075	·97983	·095	·97585
·016	·99693	·036	·98959	·056	·98415	·076	·97962	·096	·97566
·017	·99648	·037	·98929	·057	·98391	·077	·97941	·097	·97547
·018	·99604	·038	·98899	·058	·98366	·078	·97920	·098	·97529
·019	·99561	·039	·98869	·059	·98342	·079	·97900	·099	·97510
								·100	·97492

over in $7\frac{1}{2}$ or 10 seconds, can be then noticed by a practised observer with a quarter second stop watch.

In gauging rivers by means of loaded tubes, flumes are dispensed with, and marked cords may be substituted for the graduated transit timbers, being supported from the bottom if necessary, so as to be always visible; in large rivers triangulation observations are necessary. The reach should be 50 to 100 feet long, and the bottom irregularities may be removed or filled in to a certain extent beforehand, so as not to interfere with the poles, which should, when immersed, reach to about six inches from the bottom. Boats will be required to convey the poles. As the cross section will be irregular, it will be necessary to divide it into several parts, finding the area and mean velocity of each division, and calculating the corrected discharge of each division separately; the sums of these corrected discharges will then be the true discharge for the river at that spot.

6. FIELD OPERATIONS FOR GAUGING THE MISSISSIPPI RIVER AND TRIBUTARIES, BY CAPTAINS HUMPHREYS AND ABBOTT IN 1858.

Soundings.—The strength of the current, the depth and width of the river, and the floating driftwood, all combined to render an accurate measurement of the dimensions and area of cross sections a difficult operation on the Mississippi. After various experiments, the following system was adopted, by which accurate work was done even in the highest stages of the river. The middle stages were usually selected for this purpose, being preferable to the low stages, during which there would have been exposure to oppressive heat and disease, and more favourable than the high stages, when the difficulties attending accurate measurement were greatest.

Preparatory to making a cross section of the river, whether for general purposes of comparison or for determining a discharge, a base line, varying in length from 400 to 1000 feet, was measured along the bank near the water's edge; an observer with a theodolite was stationed at each extremity of this line. The one directed the telescope of his instrument across the river, so as to command the line on which the soundings were to be made; the other prepared to follow the boat with his telescope, in order to measure its angular distance from the base line when each sounding was taken. The boat, a light six-oared skiff, contained a man provided with a sounding chain, a recorder with a flag, and three oarsmen. The strongest kind of welded jack-chain was employed, to which bits of buckskin were attached at intervals of 5 feet, smaller divisions being measured with a rod in the boat. The sinker, varying from 10 to 20 pounds in weight according to the force of the current, was a leaden bar whose bottom was hollowed out and armed with grease, in order to bring up specimens of the bed of the river. The patent lead was also used for the latter purpose. The boat was rowed some little distance above the proposed section line, and allowed to drift down with the current, the sounding lead being lowered nearly to the bottom. By this precaution, the deflection of the line by the force of the current was prevented. When the first observer, stationed opposite the proposed section line, saw that the boat had nearly reached it, he waved a flag as a signal to take a sounding, and then carefully turned his instrument so as to keep the vertical hair of his telescope upon the point where the chain crossed the gunwale of the boat. The recorder in the boat, seeing the signal, waved his flag to the second engineer to follow *the boat carefully* with his telescope. The man with the

sounding chain allowed it to slip rapidly through his hands until the lead struck the bottom, when he grasped the chain at the water surface, and instantly rose to a standing position. This motion was the signal for arresting the movement of each telescope, and recording the angles. The recorder in the boat noted the depth of the water, and the nature of the bottom soil adhering to the lead. By the angles measured at the base line, the exact position of the sounding, which was never more than a few feet above or below the proposed section line, was ascertained. The process was repeated until soundings enough had been taken to give an accurate cross section of the river. Careful lines of level were then run up each bank from the water surface to points above the level of the highest floods, when such points existed, or to other convenient bench-marks. Generally, the triangles were computed, and the work plotted before leaving the place, in order to fill by additional soundings any gaps which might appear on the diagram.

At places where a series of daily velocity observations was to be made additional precautions were taken, and two independent sections, 200 feet apart, were sounded with the greatest care. Soundings, repeated from time to time upon these lines, uniformly showed that no sensible changes took place in the bed of the river. The mean of all such sections, when reduced to the same stage of the river, was accordingly always taken for the true cross section at the locality. The change in area produced by any change of level in water surface could then be readily computed from the plotted section. To determine the daily changes of this level, a gauge-rod, graduated to feet and tenths, was observed daily, its correctness of adjustment being frequently tested by comparison with secure bench-marks. An accurate know-

ledge of the area of the cross section on any given day was thus obtained. The tables of soundings for each cross section, which were all numbered, also denoted the distance of the sounding from the base line, the depth of high water during that year, and the nature of the bottom.

Velocity Measurements.—Narrow and straight portions of the river, where the form of its cross section approximated most nearly to that of a canal, where the waters of the highest floods were confined to the channel by natural banks or by levées, and where the river at all stages was free from eddies, were selected for the permanent velocity stations.

The depth and violence of the river rendered the measurement of its velocity, especially below the surface, exceedingly difficult. Of all the methods known for determining this quantity, that by double floats was found to give the best results. The method of conducting these observations was as follows:—Two parallel cross sections of the river having been made as already explained, 200 feet apart, a base line of the same length was laid off upon the bank from one to the other, being of course at right angles to both. This length was sufficient to ensure accuracy without being too great either for observing many floats in a day, or for avoiding local changes in velocity. An observer with a theodolite was stationed at each extremity of the base line. It is evident that, when the telescopes were directed upon the river, with their axes set at right angles to the base line, the vertical cross hairs marked out the lines of sounding upon the water surface, and that the time of passage of a float between these lines was that consumed in passing 200 feet. Also, that if the angular distance of a float from the base line when crossing each line of sounding was

measured, its distance in feet from the former could readily be computed, and its path fixed. Upon these principles the observations were conducted. Two skiffs were stationed on the river, one considerably above the upper, and the other below the lower section line, the former being provided with several keg floats. At a signal from the engineer at the upper station, whose telescope was set upon the upper section line, a float was placed in the river. The keg immediately sunk to the depth allowed by its cord, and the whole float moved down toward the lower line. The observer at the lower station followed its motion, keeping the cross hair of his telescope directed constantly upon the flag. At the word "mark" uttered by his companion, when the float crossed the upper line, he recorded the angle shown by his instrument, and then, setting his telescope upon the lower line, watched for the arrival of the float. In the meantime, the observer at the upper station, whose theodolite supported a watch with a large seconds hand, recorded the time of transit of the float across the upper line, and then followed the flag with his telescope. At the word "mark" given by his assistant, when the flag crossed the lower line, he recorded the line and angular distance from the base line. The float was picked up by the lower boat. By this method, the exact point of crossing each section line, and the time of transit, were ascertained. When the velocity was not too great, the time was noted by the engineer at the lower station also, to guard against error. A stop watch was sometimes used. As it was evidently impossible to observe floats daily in all parts of the cross section, the best practical method was found to adopt a uniform depth of 5 feet for all the floats, distribute them equally across the entire river, and afterwards divide the resulting velocities into groups or divisions within which the

variation of velocity was but slight; a mean relative velocity, and a mean relative discharge, for each division was then computed, the sum of the latter being an approximate mean discharge of the river, which, when divided by the area of the whole river section, gave a mean relative velocity for the whole river. The resulting discharge, when multiplied by the ratio of the velocity at the assumed depth (in this case 5 feet) to the mean velocity of the whole vertical curve, gave an accurate mean discharge of the river for that place and day.

Computation of Discharge.—A separate plot of each day's velocity measurements was made, in the following manner:—Lines were drawn upon section paper to represent the section lines, the base line, and the water edges. The distances from the base line to the points where each float crossed the section lines were then computed by a table of natural tangents, and the points laid down on the plot. Straight lines connecting the two corresponding points indicated the paths of the floats, which were of course nearly perpendicular to the section lines. The number of seconds of transit and the depth of the float was inscribed upon these plotted paths.

The diagram resulting showed that the velocities in different parts of the section increased gradually and quite uniformly with the distance from the banks until the thread of the current was reached, and, since these velocities were found to vary but very slightly for distances of 200 feet apart except in the immediate vicinity of the banks, the diagram of the daily velocity floats was divided by parallel lines 200 feet apart, the first being the base line, and the mean of all the velocities of floats in each division taken as the mean relative velocity for that division and recorded. For the shore divisions, unless the floats happened to be well distributed through them, the

mean relative velocity was assumed to be eight-tenths of that in the outer edge; a rule deduced from a subdivision and study of the velocity when thoroughly measured in these divisions.

For checking and making interpolations for the defective observations of any day in a division, the day's work was also plotted in a curve whose ordinates were the mean velocities of the different divisions, and whose abscissæ were the distances of their middle points from the base line.

The river channel being of a natural form, the sectional areas of all the divisions were unequal, and again the ratios of these areas were not constant for different stages of the river. Each divisional area was therefore multiplied by its mean relative velocity, and the sum of the products was then the mean relative or approximate discharge of the whole section; dividing this discharge by the total area of the whole section, the approximate mean velocity of the river was determined. This computation was made by logarithms, and simplified by the use of a table constructed for the purpose. In order to correct these discharges, which were those due to the velocities five feet below the surface, it was necessary to determine the value of the ratio

$$\frac{U_m}{U_5} = \frac{U_m}{U_m + \left[\frac{1}{3} + \frac{(\cdot 317 + \cdot 06f)(10r - r^2) - 25}{r^2}\right]\sqrt{bv}}$$

and multiply them by it, thus getting the true discharges, which, when divided by their corresponding areas of cross section, gave the final and correct mean velocity. The numerical values of the above expression or ratio were obtained in the following way, and put into the form of the table given.

The days on which observations were made were grouped according to even feet of the approximate mean velocities already computed, it being assumed that the effect upon the desired ratio, produced by changes in mean velocity of less than one foot, might be neglected. Each group was then examined in connection with the wind record, and days were rejected until only calm days, or those on which the wind blew directly across stream, or those on which when combined the wind effects balanced each other, were left. The resulting mean day in each group was then equivalent to a calm day, so far as wind effect was concerned. The following mean quantities were then deduced for each mean day by dividing the sum of the quantities by the number of days going to make up the mean day, viz., an approximate mean velocity of the river (v), a gauge reading, and hence a mean radius (r), and mean velocity five feet below the surface (U), found by taking a mean of the tabulated velocities of all the different divisions.

These values being substituted in the equation,

$$U = U_{d_,} - (\cdot 1856\, v)^{\frac{1}{2}} \left(\frac{d - d_,}{r}\right)^2$$

putting also $d = 5$, and making $d_, = \cdot 317r$, and

$b = \dfrac{1 \cdot 69}{(D + 1 \cdot 5)^{\frac{1}{2}}} = \cdot 1856$ when $D \ 7\ 30$; the value of $U_{d_,}$ was computed and obtained.

Next this value of $U_{d_,}$ was introduced into the same equation again to obtain new values of U, first for a value $d = 0$, secondly for a value of $d = r$, thus getting the surface and bottom velocities denoted by U_0 and U_r. Substituting for these their values in the following equation, together with those computed for $U_{d_,}$ $d_,$ and r, the value of U_m was obtained

$$U_m = \frac{2}{3} U_{d_,} + \frac{1}{3} U_r + \frac{d_,}{r} \left(\frac{1}{3} U_0 - \frac{1}{3} U_r\right)$$

Table of Ratios for correcting the approximate Discharges of the Mississippi.

Locality.	Approx. mean vel. of river.	Wind down. 4.	Wind down. 3.	Wind down. 2	Wind down. 1	Calm.	Wind up. 1	Wind up. 2	Wind up. 3	Wind up. 4
	Feet.	...	...	...	...	...	...	...	...	...
Columbus	1·6826	·90759	·92250	·93791	·95390	·97040	·98750	1·00521	1·02357	1·04262
	2·4440	·92202	·93519	·94874	·96278	·97737	·99192	1·00721	1·02294	1·03923
	3 6548	·93719	·94826	·95917	·97118	·98302	·99521	1·00767	1·02048	1·03359
	4·5097	·94400	·95407	·96428	·97463	·98546	·99641	1·00760	1·01903	1·03058
	4·3426	·94908	·95829	·96809	·97741	·98723	·99727	1·00689	1·01793	1·02858
	6·6496	·95406	·96261	·97131	·98016	·98918	·99837	1·00773	1·01727	1·02697
	7·4282	·95751	·96550	·97365	·98193	·99035	·99891	1·00762	1·01648	1·02551
	8·3162	·95983	·96747	·97523	·98311	·99112	·99927	1 00756	1·01598	1·02453
Vicksburg	3·6038	·93881	·94854	·95846	·96863	·97895	·98956	1·00037	1·01142	1·02271
	4·4110	·94544	·95458	·96423	·97340	·98310	·99300	1·00307	1·01337	1·02389
	5·5571	·95161	·96017	·96895	·97783	·98693	·99613	1·00557	1·01518	1·02494
	6·7363	·95631	·96440	·97264	·98103	·98952	·99823	1·00706	1·01604	1·02519
	7·0529					·99006				
Natchez	4·6901	·94566	·95501	·96454	·97428	·98420	·99433	1·00466	1·01522	1·02602

N.B.—A calm or wind at right angles to the current = 0; a hurricane = 10.

substituting the resulting value of U_m in the following equation :—

$$\frac{U_m}{U_s} = \frac{U_m}{U_m + \left[\frac{1}{3} + \frac{(\cdot317 + \cdot06f)\,(10r - r^2) - 25}{r^2}\right](bv)^{\frac{1}{2}}}$$

also those already deduced for v and r and b, f alone remained unknown; by giving f its value successively for each of the various forces and direction of the wind, the table at Page 107 for the stations was computed.

The approximate discharge for each day at each station was multiplied by the ratio in the table most nearly corresponding to its approximate mean velocity to obtain the true discharge, from which the true mean velocity was then obtained.

7.—FIELD OPERATIONS IN GAUGING CREVASSES BY CAPTAINS HUMPHREYS AND ABBOT.

The phenomena observed in the discharge of water through crevasses, or breaks in levées at seasons of high water, were—

1. That the effect of every crevasse, even though as large as 327 feet wide and 15 feet deep, along the line of levée, extends only for a short distance from the bank; in the above instance, it did not affect the line of motion of floating bodies passing 200 feet from the natural bank, or 300 feet from the break in the levée.

2. Between the crevasse and the outer limit of its influence there is always a movement of the water towards the break from all points below and above, which increases towards the break, and rapidly diminishes on reaching the ground in rear of the levée, where it spreads in every direction, but mostly towards the swamps.

3. There is a sensible slope along the course of this movement.

4. In passing the break, whether by a cascade or not, the water is higher in the middle of the opening than at either side.

The following was the ordinary method of computing a discharge. Knowing, from measurements made after the cessation of the flow, the high-water depth of the given crevasse, which was estimated on the line of levée, if no material excavation was made there, and on the batture in front of the levée, if holes were dug on the line of the break;—the depth on the given day was found by subtracting from this high-water depth the stand of the river below high-water mark—a quantity which was always known either from local information or from a comparison of the nearest river gauges. Taking D to represent this depth, and $W_{,}$ the maximum width of the crevasse after cessation of flow; and knowing from exact information the date of breaking of the levée, and that of the cessation of flow, the width of crevasse of any desired day could be computed; and the required discharge per second was then assumed to be equal to the continued product of this width $W_{,}$ the depth D, and the velocity (v); or $Q = W_{,} \times D \times v$; the velocity when D was less than 4 feet was taken $= 2{\cdot}818 \sqrt{D}$ (Castel's weir formula); and when D was greater than 3 feet, v was taken $= 10 - \frac{17}{D}$; the general formulæ for discharge corresponding to each case being

$$Q = (100 + \overline{n - 4}) \left(\frac{w - 100}{N - 5}\right) D\, (2{\cdot}818 \sqrt{D})$$

$$Q = (100 + \overline{n - 4}) \left(\frac{w_{,} - 100}{N - 5}\right) D \left(10 - \frac{17}{D}\right)$$

where n = number of days of discharge which have preceded the given day, and N = total number of days of discharge.

Coefficient of correction for special cases of crevasses :—

There are cases in which the conditions of the flow of water were considerably modified; such as when the levée was so far distant from the river that the depth at the edge of the natural bank was much less than that at the base of the levée; or when trees, a growth of saplings, or other obstacles existed in front or in rear of the break, both of these causing a diminution of discharge. So when the reported depth of crevasse included that of previously existing excavations on the line of levée, in these cases the resulting calculated discharge would be too high, and it then became necessary to apply in each case a special coefficient of correction. The coefficient for crevasses flowing into the Yazoo bottom was thus determined. The areas of these bottom lands and their watersheds were as follows, in square miles:—

Yazoo bottom	7110	Total. 34,600
Yazoo watershed	6740	
St. Francis' bottom	6900	
St. Francis' watershed	3600	
Tennessee and Kentucky bottom ...	750	
Tennessee and Kentucky watershed	9500	

The yearly rainfall in feet was—

At New Harmony, Indiana ...	3·92	
At West Salem, Illinois	4·02	
At St. Louis, Missouri	5·18	
Mean downfall at head of region		4·38
At Memphis, downfall for middle of region ...		4·42
At Jackson, downfall for foot of region		4·99
		feet
Mean for whole region		4·60

Giving total yearly downfall,

$= 34\,600 \times 4{\cdot}6 \times (5280)^2 = 4\,437\,126\,144\,000$ cubic feet.

To obtain the total yearly drainage, the discharge at Columbus, together with that of the Arkansas and White Rivers, was deducted from the discharge at Vicksburg; and from this also a deduction was made of the river during the year between Columbus and Vicksburg being lower by a mean difference of 6·8 feet throughout a mean width of 3300 feet for 589 miles in length; thus getting the drainage

	4 372 572 757 200	
Channel drainage ...	69 786 604 800	
Total yearly drainage	4 302 786 152 400	cubic ft.

And ratio of drainage to downfall is hence

$$= \frac{4\ 302\ 786\ 152\ 400}{4\ 437\ 126\ 144\ 000} = \cdot 96 \text{ nearly.}$$

Next, the total rainfall for the Yazoo basin, area 13 850 square miles, for from 1st December, 1857, to 15th July, 1858 = 3·64 feet × 13 850 $(5280)^2$ = 1 405 461 657 600 cubic feet; the mean rainfall 3·64 during that time being determined by register at Memphis, 3·19, and at Jackson, 4·08 feet; applying to this rainfall the coefficient of drainage before determined, the drainage from the Yazoo basin = 1 349 243 191 300 cubic feet.

The area of the Yazoo bottom was dry on the 1st December, 1857, but at high water 15th July, 1858, it had a mean depth of water of 3·08 feet over an area of 6800 square miles; having received between those dates 6800 × (5280) × 3·08 = 583 885 209 600 cubic feet, and the discharge of the channel of the Yazoo, the sole outlet, was measured during this time = 1 408 665 600 000 cubic feet. Hence, 1 992 550 809 600 cubic feet represented the total quantity which, entering the Yazoo basin between those dates, eventually drained off into the Mississippi; and the total amount of overflow from the Mississippi basin into the Yazoo basin was 1 992 550 809 600

— 1 349 243 191 300 = 643 307 618 300 cubic feet; this quantity as computed by the uncorrected crevasse formula was—

1 758 153 600 000;

hence the required coefficient of correction for the formula equals the former divided by the latter = nearly $\frac{1}{3}$. This, therefore, holds good for the crevasses in the district for which it is obtained, and the same principle can be applied to any district.

8.—SYSTEM PROPOSED BY HUMPHREYS AND ABBOT FOR GAUGING RIVERS, STREAMS, OR CANALS BY MEANS OF OBSERVED MID-DEPTH VELOCITIES.

The details of field operation to be adopted differ according to the size of the river. 1st. If the river be small and considerable exactness be required, the boat should be anchored at various equidistant stations, the banks being considered two of them, and the station actual mid-depth velocities measured by any of the known methods; the number of stations being sufficient to prevent the velocity of the water between any two of them from varying materially. 2nd. In the case of a large river, if the depth is uniform, sufficient accuracy may be obtained by observing the times of transit of a large number of double floats well distributed across the river section, the kegs being uniformly sunk beneath the surface to a depth equal to half the hydraulic mean radius of the river. Should it happen that the cross section is not sufficiently uniform and symmetrical to admit of this, the site or reach is ill chosen for the purpose. The results should then be plotted and grouped into divisions of equal width, and the mean result for each division calculated, including, of course, interpolated velocities should *any be missing.*

The depth of water in the river should be noted on a permanent gauge-post during the observations, or before and after. By this method the results obtained will be in the first case absolutely, and in the second case nearly, unaffected by the wind, no matter what its direction or force may be.

The method of computing the discharge from these observations will vary according to the accuracy required.

First method.—A close approximate result may be obtained by taking a mean of all the different station or division mid-depth velocities, and applying a coefficient of ·95 for large, and ·93 for ordinary rivers, to obtain the mean velocity of the river. In this method there are two causes of error which very nearly balance each other, namely, the inequality in area of the different divisions, and the difference between the mid-depth and mean velocities in any vertical plane, and the above coefficients meet those errors. For a rectangular cross section, no coefficient is required.

Second method.—If greater precision be required, a more accurate mean velocity of discharge of the river (V) may be computed by substituting the grand mean of all the station mid-depth or division velocities for $U_{\frac{r}{2}}$ in the following formula,

$$v = \left[(1{\cdot}08\,U_{\frac{r}{2}} + 0{\cdot}0002b)^{\frac{1}{2}} - 0{\cdot}045\,b^{\frac{1}{2}}\right]^2$$

This formula is deduced by substituting for U_m its value ·93v in the general expression,

$$U_{\frac{r}{2}} = U_m + \frac{1}{12}(bv)^{\frac{1}{2}}$$

and reducing the resulting equation.

As has been already stated, when the mean radius exceeds 12 feet, b = ·1856, and under any circumstances

$b = \frac{1 \cdot 69}{(r + 1 \cdot 5)^{\frac{1}{2}}}$. The formula therefore gives at once v the mean velocity of the river; and this simple method is quite exact in ordinary river sections, though not applicable to rectangular sections.

Third method.—Should however a very high degree of accuracy be required for testing formulæ, or constant coefficients, an amount of exactitude affected only by instrumental errors of observation may be secured by substituting the different observed division mid-depth velocities successively for $V_{\frac{D}{2}}$ in the formula

$$V_m = V_{\frac{D}{2}} - \frac{1}{12}\,(bv)^{\frac{1}{2}}$$

and the results will be true values of the mean velocities of the different divisions in terms of $v^{\frac{1}{2}}$ and known quantities. The sum of the products of these expressions, by the corresponding division areas, should be placed equal to the product of v by the total area of the cross section; and this equation, involving v and $v^{\frac{1}{2}}$ and known quantities, will give two positive values of v; the less of which, corresponding to the actual case when the velocity is greater at the axis, is the value of the true mean velocity of the river. This method, though accurate in principle, is probably not so good for ordinary purposes as the previous more simple one, which neglects the latter attempt at extreme accuracy and involves less observation, and consequently less instrumental error, as well as less labour.

9.—GENERAL ABBOT'S METHOD OF DETERMINING ON ANY GIVEN DAY THE DISCHARGE OF A LARGE RIVER THAT HAS BEEN PREVIOUSLY SURVEYED AND GAUGED.

The previous field operations consist of a survey and numerous soundings of a straight and regular portion of the channel between two bench-marks, A and B, fixed permanently near the water, whose relative levels are accurately known. An accurate plan of the river between these points is necessary, the mean cross section derived from the soundings, and a series of careful gaugings of the river on permanent gauge-posts. It is desirable that the course of the river between A and B should be as straight and regular as possible, in order to eliminate to the utmost the effect of bends, although allowances almost invariably must be made on that account. The points A and B should be well chosen, as far apart as practicable, and distant from any eddy, and be placed where the current on the bank flows with equal velocities. The latter coindtion is necessary, because water in motion exerts less pressure than when at rest, and if it moves rapidly past one bench-mark, and is nearly stationary at the other, a difference of level independent of the motive power of the stream would vitiate the observations.

On the required day the water surface at each end of the reach, A and B, has to be simultaneously referred by accurate levels to the bench-marks, to obtain the difference of level of water surface and the gauge depths. Nothing more is required. A calm day should be selected.

The formula to be used is that given in the paragraph on velocities :

$$v = \left[\sqrt{\cdot 0081 b + (225\, r, \sqrt{s})^{\frac{1}{2}}} - \cdot 09 b^{\frac{1}{2}}\right]^{\frac{1}{2}}$$

8*

the terms of which have been already explained, excepting s; in this case s is the sine of the slope of the water surface corrected for bends, and is obtained numerically by subtracting the value of h, due to effect of bends (*vide* Paragraph on Bends) from the total fall between the level stations, and dividing the difference by the total distance between them, measured on the middle line of the channel.

The method of successive approximation must be adopted to find the value of v in this formula. The following formulæ give the value of each variable in the above equation in terms of the others and known quantities; taking $Z = \cdot 93\ v + \cdot 167\ \sqrt{bv}$ and assuming $p = 1\cdot 015\ W$, should it not have been measured—

$$s = \left(\frac{Z^2}{195 r_{,}}\right)^2 \quad a = \frac{(p + W)\, Z^2}{195\ \sqrt{s}} \quad \text{and } r_{,} = \frac{a}{p + W}$$

$$\text{and } p + W = \frac{195\, a\ \sqrt{s}}{Z^2}\ ?$$

For small streams.—General Abbot modifies the above formula into the following, where v′ is the value of the first term in the expression for v—

$$v = \left\{\sqrt{\cdot 0081 b + (225 r_{,}\ \sqrt{s})^{\frac{1}{2}}} - \cdot 09\ \sqrt{b}\right\}^2 - \frac{2\cdot 4\ \sqrt{v'}}{1 + p}$$

or putting $M = \cdot 0081 b$ and $M_{,} = \dfrac{2\cdot 4}{1 + p}$

$$v = \left\{\sqrt{M + 225 r_{,}\ \sqrt{s}} - \sqrt{M}\right\}^2 - M'\ \sqrt{v'}$$

in which the term involving M′ may be neglected, for streams larger than 50 or 100 feet in cross section; and for large rivers exceeding 12 or 20 feet in mean radius M but not $\sqrt{M}$ may be neglected. The following table facilitates the application of the formula.

r.	M.	√M.	p.	M′.	Log. M′.
1	0·0037	0·0930	5	0·400	9·602060
2	0·0073	0·0855	6	0·343	9·535294
3	0·0065	0·0803	7	0·300	9·477121
4	0·0058	0·0764	8	0·267	9·426511
5	0·0054	0·0733	9	0·240	9·380211
6	0·0050	0·0707	10	0·218	9·338456
7	0·0047	0·0685	12	0·185	9·267172
8	0·0044	0·0666	14	0·160	9·204120
9	0·0042	0·0649	16	0·141	9·149219
10	0·0040	0·0634	18	0·126	9·100371
12	0·0037	0·0610	20	0·114	9·056905
14	0·0035	0·0590	22	0·104	9·017033
16	0·0033	0·0573	24	0·096	8·982271
18	0·0031	0·0558	26	0·089	8·949390
20	0·0029	0·0544	28	0·083	8·919078
30	0·0024	0·0494	30	0·078	8·892095
50	0·0019	0·0437	50	0·047	8·672098
100	0·0013	0·0369	100	0·024	8·380211

10.—THE EXPERIMENTS OF D'ARCY AND BAZIN ON THE RIGOLES DE CHAZILLY AND GROSBOIS IN 1865.

The details of the mode of conducting these experiments, which were conducted in small channels under various conditions, with the principal object of obtaining coefficients of reduction due to various surfaces of bed and banks,

cannot fail to be interesting to those intending to gauge channels of any description.

The canal of supply was Bief, No. 57, of the Canal de Bourgogne, from which the water was taken into a receiving chamber through four iron sluices, 1^m wide, and being capable of being raised $0{\cdot}40^m$, having their sills $0{\cdot}60^m$ below ordinary water level of the canal. This chamber was $5{\cdot}40^m$ wide by $14{\cdot}00^m$ long, having its bottom $0{\cdot}80^m$ below the entrance sills; the gauge sluices opening from it into the channel of experiment were of brass, twelve in number, each having a section of passage when opened of $0{\cdot}20^m \times 0{\cdot}20^m$, and having their sills $0{\cdot}40^m$ above the bottom of the chamber, and $0{\cdot}40^m$ below the sills of the entrance sluices before mentioned. These orifices resemble those of the type employed by Poncelet and Lesbros, and would, according to them, require a coefficient of reduction of discharge of 0·604, provided that the effect of the velocity of approach be neglected; in this case, however, it augmented the discharge, and an allowance had to be made on that account. The water in the chamber was constantly kept at a level of $0{\cdot}80^m$ above the centre of the gauge sluices; an appliance for showing the slightest variation of its level being continually watched by a sluice-keeper.

The channel of experiment was 450^m long before it commenced to bend towards the river Ouche; it was watertight, and was lined with planks of poplar: its fall for the first 200^m was 0·0049 per metre, and for the next 250^m was 0·002 per metre up to the bend, after which its fall to the river for the remaining 146^m was 0·0084 per metre. The different provisional constructions for employing various inclinations, and sections of different forms, were made in plank within this channel, the spaces being filled with rammed stiff earth. Nails were driven into the bottom of the channel at various points to serve as bench-marks, from

which every variation in depth of water could be obtained with exactitude. Most of the experiments were made by successively opening the twelve gauge sluices, having one fixed section and amount of supply in each case, and thus twelve results were obtained for comparison in every experiment conducted.

The velocities were principally observed by means of d'Arcy's gauge-tube, an improvement on that of Pitot; but in some cases also by floats. The latter were sometimes simple wafers, and sometimes pieces of wood or cork weighted with lead, $2\frac{1}{2}$ inches in diameter, and 1 inch thick; their times of transit over distances of from 40 to 50 metres were noted by chronometers indicating fifths of seconds, and the mean of five or more observations, in which the float following the course of the axis of the channel was adopted as finally correct.

The following was the mode of determining the measurement of discharge at the off-take.

The coefficient of discharge at the four entrance sluices was determined by closing the lower sluices and noting the time in which the former filled the chamber to a certain height; in this way the following coefficients were obtained for a head on the sill of from $0{\cdot}55^m$ to $0{\cdot}70^m$, when one single sluice was opened at a time.

Sluice raised.	Coefficient.
$0{\cdot}10^m$	0·645
$0{\cdot}20^m$	C·639
$0{\cdot}30^m$	0·631
$0{\cdot}40^m$	0·621

When the four sluices were opened at once to the full height $0{\cdot}40^m$, the coefficient was 0·637, instead of 0·621.

It was hence evident that, in order to obtain a sufficiently

constant discharge, the use of the second set of twelve sluices became absolutely necessary. The conditions of construction of the latter did not however render the contraction complete, and hence the coefficients of Poncelet and Lesbros were not applicable to them. In order to have effected this, a chamber large enough to entirely annihilate all velocity would have been necessary, the sluices should have been farther apart, and their sills should have been at least 0·60^{m} above the bottom of the chamber. It was hence necessary also to determine the coefficients of discharge for these sluices by direct observation.

In June, 1857, experiments were made with this object; a portion of the channel was closed up, and filled by opening one, two, three, &c., up to twelve sluices at a time, and the volumes thus discharged in a certain time carefully measured. The discharges per second were in these cases from 0·103 to 1·242 c.m.; and when each sluice was opened separately the discharges varied between 0·1022 and 0·1057 c.m., giving coefficients varying from 0·645 to 0·658. The irregularity of the latter was considered due to the irregularity of form of the bottom of the portion of channel filled not allowing the exact volume to be calculated: hence a mean coefficient of 0·650 was adopted provisionally for any number of sluices open at one time. In 1860, it was determined to obtain this coefficient with greater exactitude, and further experiments were made: all the practical details were carefully reinvestigated: the influence of the variations in depth of the bief or canal of supply was eventually found to exercise no effect on the irregularities; the gauge used was supplanted by a glass tube having a mouthpiece of 1 millimetre in diameter, by means of which variations in depth of water as small as 1 millimetre could be easily read. The results under these conditions were thus :—

For a discharge from	1 sluice,	the coefficient was	0·633
	2 sluices,	„	0·642
	3 „	„	0·646
	4 „	„	0·649
	5 „ and upwards to 12		0·650

For a sluice raised only 0·10^{m} instead of being fully opened, the coefficient was found to depend on the number of other sluices open, thus:—

When 1	other is opened full, the coefficient for the partly opened one is	0·650
2	... ...	0·657
3	... ...	0·660
4	... ...	0·662
5	and upwards	0·663

The determination of the coefficient for reduction for the gauge-tube.

This was effected by three methods—

1st.—By comparing the velocities obtained by means of the tube with the surface velocities shown by floats. The data according to the floats were obtained in channels 2 metres wide, having a discharge furnished by five sluices open at a time: the results gave a coefficient varying from 0·981 to 1·039 as extremes, and 1·006 as the mean of all.

2nd.—By moving the instrument at a known velocity in a mass of still water. The floats and the gauge-tube were drawn by men for a distance of 450 metres, each 50 metres furnishing a set of observations; the obliquities of the course of traction furnished the principal obstacle to arriving at a very exact result. The velocities employed varied from 0·609 to 2·034 metres, giving coefficients of reduction varying from 1·015 to 1·053 as extremes, the general mean of all being 1·034: this was considered far

too high, and the results of this set of observations were therefore entirely discarded.

3rd.—By measuring by means of the gauge-tube the velocities at a great number of points in the transverse section of the channel, and comparing the discharge calculated from these velocities with that determined by the experiments previously described; the points referred to were distributed rectangularly in vertical and horizontal lines; the discharge of each rectangle was calculated, and the sum of these discharges was employed to obtain an approximate discharge of the canal. These comparisons gave results varying from 0·968 to 1·029 as extremes, the general mean of all being 0·993.

The mean of the means obtained by the first and third methods gave a coefficient of nearly unity, which was therefore adopted for the instrument under trial.

Having thus securely determined the amount of discharge passing down the canal of experiment at any time, the levels of the water surface and its inclination being attainable also at any time with exactitude, the sectional area at any point being also known, and the coefficient of reduction for the gauge-tube being determined so exactly that any velocity observed by means of it was absolutely correct, the experiments for obtaining coefficients of discharge under different conditions, and for obtaining the ratio of the maximum velocity in a section to that of the mean velocity of discharge in open channels were undertaken.

The principal results of these experiments.

The first was the determination of the coefficient A in the formula $A = \frac{R\,S}{U^2}$ where R is the mean hydraulic radius, S the inclination of the water surface, or sine of its slope *in one metre, and U* is the mean velocity of discharge.

The coefficient was considered to vary in four categories of channel.

1st.—When the bed and banks of the channel are made of well-planed plank, or of cement:

$$A = 0{\cdot}00015\left(1 + \frac{0{\cdot}03}{R}\right)$$

the data on which this was based, are those of series No. 2 of Bazin's experiments, those of the Aqueduc des fontaines de Dijon of d'Arcy, and those of Baumgarten on the Canal Roquefavour.

2nd.—For bed and sides of ordinary plank, brickwork, or ashlar:

$$A = 0{\cdot}00019\left(1 + \frac{0{\cdot}07}{R}\right)$$

the data on which this was based, were for plank, twelve series of experiments of Bazin, and twenty-nine of Dubuat; for brickwork, the series of experiments No. 3 of Bazin; for ashlar, those of the Rigole Maréede Tillot, the Aqueduct of Cran, and the series No. 3 of experiments of Bazin.

3rd.—For channels of rubble:

$$A = 0{\cdot}00024\left(1 + \frac{0{\cdot}25}{R}\right)$$

this was based on Bazin's experiments on the Rigoles de Grosbois, and the Marseilles Canal.

4th.—For earthen channels:

$$A = 0{\cdot}00028\left(1 + \frac{1{\cdot}25}{R}\right)$$

the experiments on which this was based were those of d'Arcy and Bazin on the Rigoles of Chazilly and Grosbois, on the Marseilles Canal, the Canal du Jard, those of Dubuat on the Hayne, of Funk on the Weser, and those

of various engineers of the French Ponts et Chaussées on the Seine and Sâone.

The second result was the following formula for velocity:

$U =$ the mean velocity of discharge.

$V_x =$ the maximum velocity observed in the section.

$\frac{V_x}{U} = 1 + 14\sqrt{A}$; or $V_x - U = 14\sqrt{RS}$

or in the form most useful in the cases in which maximum velocities are observed as data for gauging,

$$U = V_x - 14\sqrt{RS}$$

Using values of A from 0·00015 to 0·003 the corresponding values of $\frac{U}{V_x}$ become thus:—

A				$\frac{U}{V_x}$
0·00015	...	...	...	0·854
0·0005	...	...	...	0·762
0·001	...	...	...	0·693
0·002	...	...	...	0·615
0·003	...	...	...	0·566

The above expression, involving terms not included in that of De Prony for the ratio of maximum to mean velocity of discharge, does not admit of comparison with it; but is evidently calculated to supersede it entirely.

The reduction of both of these results to English measures is given in Chapter I.

11.—THE GAUGING OF GREAT RIVERS IN SOUTH AMERICA, BY J. J. RÉVY.

The most recent operations in gauging very large rivers were conducted by J. J. Révy: the account of these is *given in Révy's* "Hydraulics of Great Rivers" (London,

1874), and includes a description of the method he adopted in determining the discharges of the Paranà, La Plata, Paranà de las Palmas, and the Uruguay, from which the following brief *résumé* of operations is taken.

It seems to have been a work of some time and difficulty to find a reach of the Paranà sufficiently straight for conducting gauging operations and velocity measurements; a hundred miles of the river were searched unsuccessfully, but at last a reach straight for many miles was found. Here the river was about a mile in breadth, and the soundings showed from 5 to 71 feet of water; a gauge fixed in the stream did not show a variation of level in the water surface of as much as a quarter of an inch in twenty-four hours; and the inclination of the water surface in one mile, was practically nothing.

The fall observed by levelling for one mile with a 14-inch level, on equidistant staves placed 300 feet apart, was less than ·01 of a foot; it was therefere practically impossible under the existing state of the river bank, which was not adapted for levelling, and with the instruments at hand, to carry out levelling operations with any effective result; as it would have involved ten miles of levelling on passable ground, and probably required also the use of superior instruments.

A base line of 3000 feet was measured on the low-lying left bank of the river, with a steel tape of 300 feet; and lines were set out at right angles at each end of it, to give the direction of a river-section-line for soundings; the prominent points in the neighbourhood and on the river bank were triangulated and tied into this base line.

It was found that for the surveying and triangulation work, either calm weather or clear weather with a gentle breeze was absolutely necessary;—for current observations calm days only allowed of operations being carried on.

The soundings on the lines of section were taken by the lead and cord; the length of cord was measured by a tape at each sounding, each of these measurements taking one minute, and the position of each sounding was fixed by angular observation, with a 3-inch pocket sextant giving readings to one minute, on the two flags, one at each end of the base line. The angles were observed in from three to ten seconds each. The number of soundings taken in the section varied with the necessity for them: it was necessary to show, and hence also to find the points in the river bed where there was a change of lateral slope, however many they might be, but in places where this slope was regular and gradual, the soundings were not considered necessary at closer distances than from one-twentieth to one-tenth of the breadth of the river. The section of the Paranà, where its breadth was more than 4800 feet, was sounded in two hours and sixteen minutes, after all the preliminary arrangements, drilling of the men, &c., had been properly carried out.

In plotting the section, the position of each sounding was fixed both by means of the complements of the angles observed at those points, and the calculated distances from the base.

The velocity measurements were made with the screw current meters previously described. As the velocities had sometimes to be observed at great depths, the ordinary method of lowering the meter to its position by sliding it on an iron standard was utterly impracticable, and the following mode was adopted. The current meter was attached to one end of a horizontal iron bar, 9 feet long, 2 inches wide, and half an inch thick, which was suspended by chains passing through rings attached to it from a boat moored over the required spot; in order also to prevent the current from moving the bar from its proper position,

cords from the rings of the bar were also attached to other two boats, one moored 100 yards up stream, the other 100 yards down stream. By these means the current-meter could be used with good effect in water up to 100 feet in depth, and in currents up to 5 miles an hour. Four sailors were necessary in taking current observations in this way. The observations of velocity were generally taken by an immersion of the current-meter for about five minutes, the time observed by the watch being generally a few seconds more or less, which were allowed for in the resulting calculated velocity per minute; a second checking observation was also generally made by an immersion of one minute. The instrument was put in or thrown out of gear by means of a wire leading from it up to the boat, thus allowing or preventing the revolutions of the screw from recording themselves on the dial faces at any moment.

In the gaugings carried out, observations of mean vertical velocity, giving the mean velocity in any plane from the surface of the water to the bottom, seem to have been preferred wherever practicable. For these cases, in which it was necessary that the current-meter should be steadily and evenly lowered to near the bottom and raised again to the surface, it was found necessary always to work it from a platform between two boats, placed 12 feet apart, moored by four anchors, and to have the two suspending cords marked at every 3 feet with alternately red and white marks, as guides to those lowering and raising them; the cord attached to the down-stream boat was not however considered necessary in this operation, the up-stream cord prevented the instrument from going far out of the vertical direction. In these operations the instrument was put in gear by hand by tightening a nut on immersion, and put out of gear again in a corresponding manner on withdrawal from the water. In taking surface velocity observations, the current-meter

was screwed on to a wooden staff, 3 inches wide and half an inch thick; the revolutions of the screw continuing after withdrawal from the water being at once stopped by hand so as not to vitiate the record on the dial-face.

The determination of the equation of correction for such current-meter was conducted in the following way. It was tested at a low velocity by drawing it through a distance of 189′ 6″ in the still water of a reservoir in a time of 2′ 30″ giving a velocity of 75·9 feet per minute; the average of these trials gave a recorded number of revolutions of 172, or 68·8 per minute: in the same way also it was tested at a high velocity, and showed 176·13 revolutions per minute for a speed of 183·64 feet per minute. The equation of correction being that of a straight line, two points alone are necessary to determine it: on referring these to rectangular co-ordinates on a diagram, and joining them, the true velocity corresponding to any number of revolutions of the instrument could be scaled off from the rectangular co-ordinates to the resulting straight line. Or taking it algebraically, if x and y, x_1 and y_1, be the corresponding pairs of co-ordinates for low and for high velocity,

$$\text{then } y = ax + b, \text{ and } y_1 = ax_1 + b;$$

$$\text{where } a = \frac{y_1 - y}{x_1 - x} = 0{\cdot}9962,$$

$$\text{and } b = \frac{y_1 + y - ax_1 + x}{2} = -\ 6{\cdot}811;$$

$$\text{hence } y = 0{\cdot}9962\,x - 6{\cdot}811,$$

or in the form more useful for obtaining the true velocity, x, from the number of revolutions, y,

$$x = 1{\cdot}00381\,y + 6{\cdot}837.$$

On applying to this equation a value of $y = 0$, we obtain

as a result that this particular instrument would cease to record revolutions for a velocity of less than 6·837 feet per minute.

Current Observations.—In consequence of the rivers observed being tidal, and having a variable current, it was necessary to moor a permanent observatory at a convenient point in the deep part of the river on the line of section, and make hourly observations of the current from it throughout the day and night. The tidal rise and fall was also registered at every quarter of an hour; barometric, thermometric, and wind observations were also recorded.

The current observations, both surface, mean, and sub-surface, were taken with Révy's current-meter from a small boat moored temporarily fore and aft on the line of section already sounded, its position in each case being determined by angular measurement with a pocket sextant on the extremities of the base line, which fixed it within a few inches. For this work two sailors, two anchors, and several hundred yards of line were necessary. The current observations were taken at the surface, and at depths of 4, 7, 10, 16, and 23 feet, the latter being one foot above the bottom. The mean current observations were made three times in each case, and were found to check each other within 1·6 foot per minute in observations giving 80 feet per minute. The time of day of the current observations was always noted, and check observations were also taken from a fixed level, so that the observed tidal variation might be applied, and the effect of the tidal wave—a disturbing cause far greater than that due to the inclination of the water surface in the cases of these rivers—thoroughly investigated.

A convenient mode was adopted for testing the straight-

ness of the reach of the river at the section in which the velocities were observed. The centre of gravity of the river section was found and marked on the drawing, and also the centre of gravity of a section whose depths represented the surface currents in any convenient mode, either feet per minute or per second; the horizontal distance apart of these two centres of gravity indicated the amount of effect of a bend in the reach at that section. In the Rosario section of the Paranà this was $\frac{1}{273}$ of the width of the river, and the section was considered favourable; in the Palmas section it was as much as $\frac{1}{25}$ the width of the river, and this was not considered favourable. In cases where a very straight reach is not to be obtained, the position of a section of observation is recommended to be taken at the point of contrary flexure of two reaches curving in opposite directions.

The conclusions arrived at by M. Révy from his study of the current observations on the La Plata, Paranà, Paranà de las Palmas, and Uruguay, were—

1st. That at a given inclination surface currents are governed by depths alone, and are proportional to the latter. 2nd. That the current at the bottom of a river increases more rapidly than at the surface. 3rd. That for the same surface current the bottom current will be greater with the greater depth. 4th. That the mean current is the actual arithmetic mean between that at the surface and that at the bottom. 5th. That the greatest current is always at the surface, and the smallest at the bottom; and that as the depth increases, or the surface current becomes greater, they become more equal, until in great depths and strong currents they practically become substantially alike.

12.—GENERAL REMARKS ON SYSTEMS OF GAUGING.

The foregoing brief accounts of the modes adopted by various hydraulicians in carrying out field operations form a far better guide to the engineer about to undertake the execution of gauging operations than any arbitrary advice, or set of rules, could possibly be; the author may, however, be permitted to make a few remarks in conclusion. It is, of course, assumed that the most advisable mode of proceeding in one case might not be applicable to another, and that the method of gauging should be suited to the general object, the place, and the circumstances. When the object is of an experimental nature, having scientific results in view, the experimentalist himself is the best judge of the mode most suited to his object. Most gauging operations, however, have for their purpose the determination of the discharges of a river, or of canals, with as little labour and expense, and in as short a time as anything approaching to accuracy of result will admit; in these cases the amount of accuracy required is that which fixes the mode to be adopted.

1. The most rapid and least accurate mode of determining the discharge of a river or canal at a certain place and time is that which dispenses with velocity observations, and makes use of a calculated velocity formula as a substitute. The dimensions of two parallel sections of a straight reach of the channel are measured, the inclination of the water surface between the two is levelled, and the nature and quality of the bed and banks are noted; these data enable the discharge to be calculated by the aid of the most modern and most correct formula with a certain amount of approximate truth. The point now to be considered

is what amount of exactness may be reasonably expected from the practical application of this method.

The Kutter formula for mean velocity of discharge (for metres),

$$V = c_{\prime}\sqrt{RS} \text{ ; where } c_{\prime} = \frac{z}{1 + \dfrac{x}{\sqrt{R}}}$$

$$z = 23 + \frac{1}{f} + \frac{\cdot 00155}{S}; \text{ and } x = f\left(23 + \frac{\cdot 00155}{S}\right)$$

seems theoretically to leave nothing more to be desired, except perhaps a simplification of form not attainable in the present state of hydraulic science. It is applicable to channels of all dimensions, from the smallest distributary or rigole to that of the Mississippi; and can be applied to channels of any material, from weed-covered earthen beds to cut stone and carefully planed plank, the data on which it is most carefully based being those of numerous experimentalists. The functions or terms involved are only three, R, S, and f, of which the two former can in most cases be readily and sufficiently exactly observed in practice; the great difficulty, however, lies in the determination of the third function. An examination of the general and the local values of f, given at page lxix. of the Working Tables, will explain this. Among the general values suitable to beds of special construction, from well planed plank to rubble, the value of f ranges from 0·009 to 0·017; and the gradations of roughness or quality of surface are clearly marked by the corresponding values of f, the greatest gap being the difference between 0·013 for ashlar and 0·017 for rubble, a difference that can be easily worked up to in practice without any likelihood of important error. It would hence appear that there would be no difficulty in practice of determining discharges with *fair* accuracy by means of the above calculated velocity

formula for channels constructed in such artificial materials. It is, however, in the cases more usual in practice, namely, in those of canals having earthen beds and banks, and in natural river channels, that the values of f offer so wide a range of choice, that the calculated discharge might involve serious error as the result of the adoption of an unsuitable coefficient. For earthen canals the values of f range from 0·020 to 0·035, the gradations of which are far from being yet sufficiently definitely marked; and for local values the range is about the same. It would seem, therefore, that in these cases it would be necessary to determine by velocity measurement the discharge of the river or canal under consideration, and thence deduce a value of f suitable to it before the above method could be applied for obtaining its discharge at any time or place with sufficient accuracy; or, in other words, the actual gauging must be done before this mode of procedure can be adopted. In the future we shall probably have the values of this function more definitely laid down, and we shall then be able to make use of this method more readily, and with greater confidence in the results; now we have only the present amount of information to guide us, and are hence unavoidably forced into a certain amount of velocity measurement as a means of correctly gauging canals and river channels in earth.

2. Assuming, therefore, that velocity measurement is absolutely unavoidable, the question next arises, what is the least amount of it necessary in determining a discharge? The results of Bazin, determining the relation between the maximum velocity in a section and its mean velocity of discharge, give the readiest solution of this problem. His formulæ are for metres,

$$\frac{V_x}{U} = 1 + 14\sqrt{A}\,;\ \text{or}\ V_x - U = 14\sqrt{RS}$$

where V_x = the maximum velocity, and U = the mean velocity of discharge; and it is evident that by combining with this formula the more modern coefficients of Kutter, we can with the aid of only a few observations of maximum velocity, arrive at a mean discharge with rapidity, and a fair amount of accuracy, and be afterwards able to determine a discharge at any time under the same local conditions by means of the ordinary calculated velocity formula and the Kutter coefficient already mentioned, without the use of more velocity observations. The reduction of these equations to English measures is given at page 33, Chapter I.

It is extremely probable that this mode of gauging will be more universally adopted in future, and that a large series of observations will throw more light on the relation of the maximum velocity to the mean velocity of discharge, and enable it to be determined with greater accuracy than is at present possible. Observers are therefore recommended to keep in view in all gaugings conducted on this principle, not only the sectional position of the maximum velocity in a section, which may be confined to a single point either in the middle of the channel at the surface, or at a few feet below it, around which the velocities may diminish in section rather suddenly, or may extend with but little diminution over an important portion of the section, but also the locus of maximum velocity, or its depth below the water surface, which may vary sensibly in a long reach of river; this inclination of the locus, as well as the amount of section of very high velocity, being data that will probably aid eventually in determining the ratio of maximum to mean velocity of discharge with greater precision.

3. The next mode of gauging that seems most applicable to ordinary rivers is one of the modes recommended by Captains Humphreys and Abbot. This, however, involves a

greater amount of velocity observation, and at the same time requires the velocities to be observed at a greater depth, for which all descriptions of current-meters are not applicable.

The velocities are all observed at a uniform depth equal to half the hydraulic radius of the section, and at equal distances judiciously chosen across the line of section; and the mean of these velocities $U_{\frac{r}{2}}$ is taken;—the mean velocity of discharge, v, is then obtained in the formula,

$$v = \left[\left(1{\cdot}08\, U_{\frac{r}{2}} + {\cdot}002\, b \right)^{\frac{1}{2}} - {\cdot}045 \sqrt{b} \right]^2$$

where $b = \dfrac{1{\cdot}69}{(r + 1{\cdot}5)^{\frac{1}{2}}}$.

This mode should, however, be limited to ordinary and large rivers; in fact, the application of any of the Mississippi data or formulæ to artificial channels or small streams cannot be recommended.

4. The next further attempt at accuracy in river gauging involves a complete investigation of the whole of the velocities in the channel section; the velocity at every point in the cross section should be known and plotted on a diagram, they can then be grouped into divisions of the section by vertical and horizontal lines within which the variation of velocity is not important: a mean velocity for each division is calculated and multiplied by the area of that division to obtain its discharge; the sum of these discharges is the discharge of the whole section. Such detailed observations when carried out on an extended scale involve a large amount of labour, care, and skilled personal superintendence, but at the same time afford results not only valuable as regards the determination of the discharges of the river specially under consideration, but also as records of hydraulic experiment aiding in the progress of science.

CHAPTER III.

PARAGRAPHS ON VARIOUS HYDRAULIC SUBJECTS.

1.—ON MODULES.

HYDRAULIC engineers not having yet arrived at a perfect module for measuring the amount of water drawn off in an open channel for irrigation or other purposes from an open canal or reservoir, under a varying head of pressure, it is a matter of some interest to examine the older types of design of modules that have been used at various times, and in various countries, before going on to those of more modern form. The designs being necessarily simple, they will be found perfectly comprehensible by means of description without the aid of drawings or diagrams.

Piedmont appears to have been the birthplace of modules, for although irrigation is essentially Oriental in origin, owing to its extreme reproductive power in hot climates, and though it was introduced into Europe by the Moors, we do not find, either in India or in Spain, where portions of these works still exist, anything approaching to a module. The systems employed in carrying out irrigation almost prove that they had not such a thing at all. In India the practice seems to have been to turn water on to a field until either the landowner or the turner-on of water was satisfied, or perhaps rather until the landowner was satisfied that he could get no more. No doubt this was the best plan to start with, as the object of irrigation was to water the fields sufficiently, and the *landowner being* the best judge as regards how much water was

required for his crop, this mode insured the observation of the proper persons. This plan was, however, open to one very serious objection; when the landowners discovered that an extra amount of water beyond that strictly necessary for the crop was in some cases capable of increasing the amount of produce to a small degree, they would take more water, either by stealth or otherwise; the amount of perpetual squabbling on this subject would then have been very large, had it not been for the fact that in Oriental countries irrigation works were made by rajahs, emperors, or chiefs, whose despotic rule and despotic institutions supplied a very practical limit in such matters—moral or physical force.

In Spain, under Moorish rule, it is probable that this useful substitute for modules was also in vogue; but in the huertas or irrigated lands of Spain in more modern times and under Christian rule, the water being the joint property of several villages that combined to keep the works in order, and legislated for themselves about the distribution of the water, the first great step, the just division of the water on a large scale among the several villages, had to be regularly carried out. The canals being comparatively small, a proportional division was effected by equalizing the size of a certain small number of outlets from the main canal into the subsidiary channels, one village thus taking a fourth or a sixth of the total volume of water passing down the canal.

In Piedmont the conditions were different; the country being hilly, and the water taken from streams and torrents having a considerable fall, water power was extensively used for driving corn mills. It is probable that there were a few water-driven corn mills both in India and in Spain, but there such mills would be public institutions, the miller being a servant of the community, generally living on a fixed income, or yearly pay, given either in kind or in money by all the neighbouring villages using the mill. In Piedmont the mills were the private property of individuals, as they are at the present day in Europe; hence it was there that the first unit of water measurement was arrived at—the amount of water enough to drive a corn mill, which were probably then and there of about the same size and requirements. This amount of water then assumed a technical name, the *ruote d' acqua;* the same thing in Lombardy being called a *rodigine*, in Modena a *macina*, and in the

Pyrenees a *moulan*—the same circumstances in various places leading to the adoption of a similar unit of measurement, which was naturally rather variable. In Piedmont the amount was generally about 12 cubic feet per second, and was supplied by an outlet 19 in. to 20 in. square, the water issuing free from pressure at the surface level. The next step was the introduction of a smaller unit of measurement for purposes of irrigation for discharges under pressure, the Piedmontese *oncia;* which was a rectangular outlet 5·04 in. broad, 6·72 in. high, having a head of water 3·36 in. above the upper edge of the outlet; its discharge was 0·85 cubic feet per second, and this was the immediate parent of the Piedmontese module, and, as far as we know, the ancestor of all modules.

Piedmontese Modules.—These, the most perfect type of which is that of the Sardinian code, were designed or intended to fulfil the following conditions: that the water should issue from the outlet by simple pressure, that this pressure should be maintained practically constant, that the outlet should be made square in a thin plate having vertical sides, that the issuing water should have a free fall, unimpeded by any back-water, and that the water of the canal of supply should rest with its surface free against the thin wall or stone slab in which the outlet was formed. The following is a description of the general type. The water is admitted through a sluice of masonry, having a wooden shutter working vertically, into a chamber in which the water is supposed to lose all its velocity and is kept to a fixed level mark by raising or lowering the shutter; the chamber is of masonry and has its pavement on the same level as the sill of the sluice, the regulating outlet from this chamber being an orifice 7·854 in. square, having its upper edge fixed at 7·854 in. below the fixed water-level mark of the chamber. Its discharge is 2·04 cubic feet per second. If a larger discharge at one spot be required, the breadth of the outlet is doubled or trebled, the other dimensions remaining unaltered. Such are the sole unalterable conditions or data of this module; all its others seem to have varied very greatly; its sill is sometimes at the level of the bed of the canal of supply, sometimes above it, and sometimes below it, in which case a slight masonry incline was made from the bed down to it; the length and breadth of the

chamber vary greatly, the former from 15 ft. to 35 ft., its form being circular, oval, or pear-shaped; the side walls splaying outwards sometimes close up to the sluice, sometimes not till near the regulating outlet, the object being to destroy the velocity of the water within the chamber. The lower edge of the regulating outlet is generally, but not always, placed at 9·825 in. above the floor of the chamber. The paved floor of the chamber is in many cases, but not in all, continued at the same level beyond the outlet.

The practical advantages of this type of module consist, therefore, in having a chamber in which the water can be kept to a constant level, and from which the water can issue under a constant head of pressure through a regulating orifice of fixed dimensions.

Milanese Modules.—The *modulo magistrale* of Milan is the most improved type of Lombardian modules, the *modulo* of Cremona and the *quadretto* of Brescia being very inferior to it in design; its principal advantage over the Piedmontese modules being the fixity of dimension of almost all its parts; in other respects it resembles it very much, the principal differences being that the water chamber is always rectangular and covered with slabs, and is hence called the covered chamber, that its flooring has a reverse slope in order to deaden velocity, and that the masonry channel beyond the regulating outlet has fixed dimensions also, a portion of it being called the outer chamber. As to its general arrangements, the sluice of supply has its sill invariably on a level with the bottom of the main canal, which is paved with slabs near it; the breadth of the sluice is the same as that of the regulating or measuring outlet; the sluice gate is worked by lock and level, being fixed and locked at any required height by catch lock and key. As to dimensions, the covered chamber is 20 ft. long, its flooring having a rise of 1·75 in. in that length, and its breadth is 1·64 ft. more than that of the sluice of supply, that is, 82 ft. more on each side; the lower surface of its covering of slabs or planks is fixed at 3·93 in. above the upper edge of the regulating outlet, which is the height to which the water must be kept to secure the fixed discharge. In order to gauge the water in the chamber, a groove is made in the masonry so as to allow a gauge rod to be introduced within at the sill of the sluice, which will read 27·51 in. of water above the sill, when the

proper head of pressure exists; should it read more or less, the sluice gate must be raised or lowered. The outer chamber is 7·86 in. wider than the measuring or regulating outlet, its total length 17·79 ft.; its side walls, which like those of the covered chamber are vertical, have a splay outwards, so that the width at the farther end is 11·72 in. greater than at the outlet end, that is to say, it is there equal in width to the covered chamber. To insure a free fall, the flooring of the outer chamber is 1·96 in. below the lower edge of the outlet, and has besides a fall of 1·96 in. in its length of 17·72 ft.

The total length of the module is nearly 37·75 ft., but its breadth is variable, according to the amount of discharge required. If intended to discharge a Milanese *oncia magistrale*, the Milanese unit, which varies from 1·21 to 1·64 cubic feet per second according to different computations, averaging, 1·5 cubic feet per second, the measuring outlet is 7·86 in. high and 4·12 in. broad, under a constant head of pressure of 3·93 in.; the breadth of the covered chamber being 25·54 in., and the breadths of the open chamber 13·75 in. and 25·54 in.

It is essential to the effective operation of the regulating sluice that the difference of level between the water in the canal and that in the module be at least 7·86 in.; and as the height of water in the latter must be 27·51 in., the depth of water in the canal must never be less than 35·37 in. or 3 ft., in order to allow the module to work properly. The following are the relative levels of the parts of the module, referred to the bottom of the main canal as a datum:

	Inches.
Water surface in the interior of the module	27·51
Upper edge of the measuring outlet ...	23·58
Upper end of flooring of open chamber ...	13·75
Lower end of the same	11·79

Such is the type of the Milanese modules, the dimensions being suitable for a discharge of 1·5 cubic feet per second; unfortunately, in point of fact, the type has been rarely rigidly adhered to, and thus its advantages as a universal, or even as a local water standard have been comparatively thrown away in practice. Its use, however, established a discovery that was at that time very important,

viz., that larger outlets gave a greater discharge than that due to the proportion of their section for small ones; it was therefore determined that no single outlet of a module should be made for a discharge of more than eight oncia or 12 cubic feet per second; and when a greater discharge was required, two or more separate outlets were to be used in combination. A gauge post was also found to be necessary in order to enable the water guardians to adjust the sluice accurately.

The principal defect of the Milanese modules is that, owing to the rush of water from the canal, it is nearly impracticable to keep a constant head of pressure on the measuring outlet; besides this, sand and fine silt vitiate the accuracy of amount of discharge.

Such are the comparatively ancient modules, the Milanese *modulo magistrale* being the most improved one of them. Their type has been very much adhered to in modern times; that of Messrs. Higgin and Higginson on the Henares Canal may be considered as the greatest improvement that can be made on them, without departing from that type. In this module, the entrance by a sluice into a chamber for destroying velocity has been preserved, but the exit is an overfall, and hence more susceptible of exact measurement of discharge; the means applied to deaden the velocity of entrance are again different.

The entrance into the channel through a wall is a passage 23·6 in. (·6 metre) square, regulated by a well fitting cast-iron door raised by a screw; the chamber is rectangular, 10·37 ft. long, by 7·20 ft. wide below, 9·20 ft. above, the side walls having a batter of 1 in 6. The bottom of the chamber is horizontal and at a level 72 feet below the sill of the entrance sluice. To deaden the action of the water, a partition of masonry grating is built across the chamber at a distance of 4 ft. from the wall, and 5 ft. from the overfall wall of exit, it is 1·37 ft. broad, and has eight slits or vertical passages not cross-barred, each slit being 5·4 in. wide. The water having been deprived of all action by passing through this arrangement, enters the second portion of the chamber, and then passes over a weir having an iron edge 6·56 ft. (2 metres) long, fixed nearly on a level with the top of the entrance sluice, or 2 ft. above its sill. The discharge required for irrigation being never to exceed 176 litres or 6·22 cubic feet per second, the depth on the

weir sill will therefore never exceed ·5 ft., the sluice opening being 1·97 ft. square.

There are two small side walls having a batter from above on either side of the sluice entrance, these walls projecting into the main canal, in order to protect the entrance and prevent silt from accumulating there, which otherwise, and perhaps even in any case, would have to be dug out occasionally. In order to keep the chamber in proper working order, a keeper must be employed, and a gauge post erected in the canal, with reference to which he lowers or raises the sluice, and keeps the water in the chamber always at a fixed level.

It is evident that the changes may be rung on this species of module to a great extent without effecting great improvement, by increasing the number and altering the positions of the sluices and overfalls, and modifying the arrangement for deadening the action of the water. This has been done in many cases without much result; it is hence not worth while to bring forward other examples of this type.

Although some of these are complicated in form, as well as much varied in detail, the types are exceedingly simple; they all require the occasional attendance of a keeper for adjusting them according to the variation of pressure; they are made of brickwork and masonry, and consist of a series of open passages and covered chambers connecting orifices and overfalls. It is quite evident that, except under special circumstances, such modules are far behind the wants of an age that economizes labour, attendance, and supervision wherever possible.

Self-acting Modules.—A module to be of any use now must in the first place be self-acting. Nor, indeed, is this all. A large number of self-acting apparatus for regulating the supply or flow of water have been designed and used, but three-quarters of them do not answer all the purposes required of them at present. Some are large, some expensive, others involve a large expenditure in protective or additional large chambers, others are complicated and liable to get out of order, and others involve a great loss of head, which, in the case of their application to irrigation canals of small fall, is an insurmountable objection. The worst of them may be

said to be those that fail in their main object in producing practical accuracy of discharge. With all these objections to deal with, it will not be necessary to do more than make passing comments on the greater number of them, and the principles involved in their design and construction.

We will, however, first mention the requirements of a good module. The primary consideration is that under all ordinary circumstances the discharge may be practically constant and correct, that is, should not be liable to vary more than 5 per cent.; secondly, that it should be very simple in construction and application; thirdly, that it should not be liable to derangement; fourthly, that it be portable, easily applied and removed from any portion of the canal without involving much waste or loss; fifthly, that it should not involve much loss of head, and that it should be able to drain the main canal or basin of supply, down to a level of one foot above its bed, and deliver water if need be as high as within one foot of full level in the canal; sixthly, that it be inexpensive, not costing in England more than about 10*l*., and more than £5 additional for its attachments, slabs, cisterns, or chambers, and setting it in place in working order.

There are perhaps only three modules yet designed that may be said to fulfil these conditions; these we will for the present term portable modules, and defer dealing with them until after commenting on the others, or ordinary self-acting modules, some of which have advantages or disadvantages worthy of notice, or have attracted special attention in any way.

Until recently, the power of flotation was the sole means adopted in self-acting modules for obtaining an equal discharge under varying heads in the canal or basin of supply. The simplest manner of applying this is perhaps in attaching or fixing the pipe or pipes of supply to the float itself, thus insuring a fixed head of pressure on their entrance, however much the surface level in the supplying basin may vary. So far as this, the modules depending on this principle appear excellent, but unfortunately all of these seem defective on account of other considerations. For instance, in "*the suspended opening,*" where the water enters through two horizontal pipes into the body of the float itself (which is kept submerged to a sufficient depth by weights) and passes out of it through a vertical

pipe fixed on to the lower side of it, the vertical pipe has to slide up and down in a species of stuffing-box in a masonry platform below, so as to discharge itself clear of the water in the main canal, and prevent the latter from leaking through into the well below the platform, from which the moduled water alone should be drawn off. This is plainly a contrivance that would be defective for purposes of irrigation; should the vertical pipe not slide easily into the stuffing-box, the power of flotation may be entirely neutralized; should it be too easy there will be leakage, and perhaps to a serious amount; the loss of level is seriously great, the delivery level never being higher than 1 ft. above the bed level of the canal. Modifications of this contrivance, having in view the abolition of the loss of head, have been made by using syphons either erect or inverted, instead of the sliding vertical pipe. They certainly attain that object, but introduce new defects sufficient to render them less useful for purposes of irrigation than the original suspended opening; they are expensive, and difficult to manage, the action of the syphons is liable to be stopped by accumulation of air, and their discharge is not only practically low in comparison with their theoretical calculated discharge, but also is variable, as they are very liable to foul; their adjuncts, chambers around and attached, are expensive. The vertical pipe arrangement of the suspended opening is the principle on which many water-meters, used by water companies for discharging water in large quantities, have been constructed.

The same principle has been adapted to purposes of irrigation in the module of M. Monricher, on the Marseilles Canal, constructed between 1839 and 1850: it is intended to supply irrigation channels having discharges of from 1·06 to 4·24 cubic ft. (30 to 120 litres) per second as a constant supply. The details of construction are as follows: A masonry reservoir 11·15 ft. by 14·76 ft., having its bottom at a level approximately 3 ft. below the bottom of the canal, is connected with it by a rectangular masonry passage having a horizontal masonry covering at the level of low water surface in the canal; a transverse masonry wall stops the action of the water, which enters the reservoir afterwards by two passages, one on either side, the wall and passages taking up a portion of the reservoir space. Beyond two pairs of grooves for putting in stop-*planks* for shutting off the water entirely during repair, there is no

other sluice or check to the free flow of the water. In the centre of the rectangular reservoir is a cylinder of masonry, having an internal diameter of 2·30 ft., being 1·00 ft. thick, the bottom of it being approximately 2·00 ft. below the bottom of the reservoir, and its top edge about 2·00 ft. below low water canal surface. An iron cylinder is made to fit the internal masonry closely, and to slide up and down it, and to hang by a rod and adjusting screw to a wooden bar supported by two wooden floats placed clear of the masonry, each of which is 1·64 ft. deep, 1·31 ft. broad, and 5·24 ft. long. There are also two vertical bars in the reservoir outside the floats, up and down which the bar slides on rings. The adjusting screw enables the iron cylinder, which is about 5·8 ft. long, to be placed so that its upper edge may be set at any depth below the water surface, so as to produce any required discharge. This, when once fixed and checked, is never altered. The whole is enclosed in a locked building.

The water of the reservoir therefore enters the iron cylinder above, and flows out below; the lower water being divided from the rest of the reservoir above by masonry partitions, it rises through the masonry passage thus made into the masonry water-course or irrigation channel, the bottom of which is not more than ·75 ft. below that of the bed of the main canal; the channel section is 2·00 ft. by 1·31 ft., having a small enlargement 3·28 ft. square at the commencement of the channel. Plans and details of the module here described are given in Moncrieff's "Irrigation in Southern Europe."

In this module, therefore, the section of outlet, viz., that of the iron cylinder, is constant; the edge of the cylinder rises and falls by flotation; the loss of level is as small as can be conveniently obtained in modules of this principle of design, and if the cylinder could, without much care or superintendence, be made to work well in the masonry without leakage or friction to any detrimental extent, as stated by the engineers of the Marseilles canal, the amount of inaccuracy of discharge cannot be great. It would doubtless be an improvement were some arrangement applied to this module for preventing silt from entering the reservoir, which must be liable to interfere with the working of the cylinder, and produce a greater deteriorating effect in this module than in many others. The

masonry portion of the module would require good workmanship, and the putting together of the whole in good working order considerable care. It is, therefore, rather expensive, and certainly has not the element of portability.

The Suspended plug is like the suspended opening, a principle that has been adopted for modules and applied in a very large variety of ways, some of which involve complexity of parts and details. Its main principle is probably slightly more modern than that of the latter: both are decidedly old, but as these old contrivances are perpetually being re-invented, a brief description of their principles may be of use to some, while comments on them may deter others from wasting their energies on an idea that appears to have been fully worked out.

The simplest case of the suspended plug is this. A circular orifice is fixed in a floor at the level of the bed or bottom of the canal or reservoir, and a plug of varying section is suspended in it, being attached to a float that rises and falls with the surface of the water; the annular water passage thus left open is made to discharge equal quantities under varying heads by proportioning the section of the plug throughout its length; the area of the annular opening being in inverse proportion to the velocity of discharge. To insure a free fall there is a well below the floor into which the water falls to a depth equal to that of the depth of the floor from high-water level of the canal. The depth of the float and its attachment to the plug prevent its acting at a depth of water of less than one foot in the canal. These two points, which are serious objections to the adoption of this module on irrigation canals, have been much modified in the more complicated modules constructed on this principle, which will hereafter be mentioned. As to the plug itself, it is either a conoid hung in a circular orifice, or a flat-sided conoid of equal thickness in one direction hung in an orifice which is rectangular laterally and of circular curvature transversely; in the latter case a fixed area is left open on the flat sides of the plug which has to be allowed for in the calculations for the section of the plug. The diameter of the plug in the case of the conoid is obtained by calculating the areas required to pass the required *discharge for* various heads of water, as, from 1 to 10 ft. for every

three inches, and deducting these from the fixed area of the orifice, the remainders are then the areas of the circular sections of the plug for those depths from which the diameters are obtained. The flat conoid can be made of the same lateral section for all discharges, the thickness of the flat sides being increased in direct proportion.

The following is an example of a module designed on the suspended plug principle, and is perhaps the simplest application of it in actual practice. It was designed by Don Juan de Ribera, projector of the Lozoya canal, or canal of Isabella Segunda, and is used on that canal with good effect.

It is so arranged that the size of the outlet diminishes when the head of water increases. The module itself is a long tapering bronze plug, ·524 ft. in diameter at its lower end, and is attached to a circular brass float above, which floats freely in the water of a masonry well 3·38 ft. by 3·94 ft. square and 4·16 ft. deep; at the bottom of this well, which is on a level with the bottom of the main canal and the rectangular masonry passage connecting them, is a circular orifice 1·56 ft. in diameter, within which the lower end of the module is made to work vertically, the plug and plate being of bronze to prevent rust. Below this well again is a second one, into which the water falls after having passed through the ring between the orifice and the plug. The entrance of the rectangular passage leading from the canal, which is only about 3 ft. long, is protected from silt by an iron grating, and is covered in at the top by slabs to the full level in the canal; the well is also covered in by a locked iron trap-door. In this module friction is reduced to a minimum; the module hangs freely from the centre of the float, and can be slightly raised or lowered in order to diminish or increase the discharge passing through the ring or space between the edge of the orifice and the plug; but when a constant discharge is required it is finally properly adjusted, and then entirely left alone. The float is about 2 ft. in diameter, having a thickness in the middle of about ·9 ft., and at the edges of ·6 ft.

This module discharges one cubic metre (35·3166 cubic feet) per hour, and is hence styled an horametre, the discharge being ·2777 litres, or ·0098 *cubic feet per* second. The curve of the module or

bronze plug is such, that the roots of the vertical abscissæ vary inversely as the differences between the squares of the radius of the orifice and of the horizontal co-ordinate. Hence, if the required discharge is given with a head of water of one metre, when the diameters of the orifice and plug are respectively ·20 and ·1653 metres, then, if the head of water be reduced to ·81 metres, the diameter of the plug at the level of the orifice must be ·1610 metres, as

$$\sqrt{1} : \sqrt{\cdot 81} : : (\cdot 20)^2 - (\cdot 1610)^2 : (\cdot 20)^2 - (\cdot 1653)^2.$$

The lengths corresponding to the different diameters of the taper of the plug will, for a constant diameter of orifice of ·20, be as follows :—

Depths from water surface	·10	·12	·16	·41	·77
Diameters of plug	·00	·0585	·0912	·1211	·1374
Depths from water surface	1·26	1·90	2·71	3·71	
Diameters of plug	·1480	·1554	·1610	·1653.	

The principle being that the velocity of discharge through an orifice varies with the square root of the head of water; thus, taking R r to represent the radii of the orifice and plug respectively, the discharge per second

$$Q = c \pi (R^2 - r^2) \sqrt{2 g H},$$

H being the head of water, the value of the experimental coefficient, c, being for this case deduced, from a series of experiments of Don Juan de Ribera, to be ·63, in accordance with similar results obtained in ordinary practice in parallel cases. This is probably the module in most perfect accordance with theory yet designed; it is, however, of small dimensions, and hence likely to be much affected by even the very small proportion of silt that would pass through the grating. Its principal defect is, that the loss of level necessarily involved in it in order to obtain a free fall would render it inapplicable in a very great number of cases, where even a few inches of fall are of extreme importance.

The modifications of this type of module consist in putting the float in a separate chamber, which thus becomes a silt trap, and relieves the orifice from being affected by silt, the connection between the float and the cone being either a chain passing over

two runners or a lever: in these cases the plug is reversed, having its broader end upwards; the friction involved affects the working of the module and its accuracy of discharge, and, in the case of levers, the lengths of the arms modify the quantities employed in the calculations of sections of discharge. In some cases the form of the lower well assumes various forms, having for their object the reduction of the loss of level existing in the more simple type. It is extremely doubtful whether any of these modifications can be considered advantageous on the whole.

Rising and Falling Shutters.—Contrivances of this type are generally suited for large quantities of water where great accuracy is not required. The falling shutter, as used on canals in England or Scotland, is an oblique shutter hinged below, and raised or lowered in front of an opening in the side of the canal by two floats in recesses, the water passing over the upper edge of the shutter in a tolerably uniform volume. The rising shutter is a vertical shutter in front of an opening in the side of and down to the bottom of the canal; it is raised or lowered by means of a float attached to it by a chain passing over a runner, the float being in a separate chamber, and having trunnions and friction rollers running in curved grooves or recesses on each side of the chamber; these curves require very accurate construction in order that the discharges may not vary under different heads. Shutters of this description having pressure on one side only are very liable to stick, and get out of order; they are hence very inferior in practice, although new ones under favourable conditions can be made to work very accurately.

The above three types comprise the whole of the non-portable self-acting modules that have been much used in practice to good effect.

Portable Self-acting Modules.—In this class we comprise such modules as could be removed or replaced without much difficulty or loss. There are three such modules that have attracted attention, though there are probably others not so well known.

The first is that of Lieutenant Carroll, of the Royal Engineers: its principle is exactly that of the well-known draught regulator: the pressure of the water is made to regulate the opening in the

one case in the same way as an increased draught of air is made to partially close the opening in the other; and the application of the principle is excellent for the intended purpose—it can be made almost entirely of iron, is simple, effective, and admits of removal without causing much loss or expense. Drawings of this module are given in the Rurkhi Professional Papers.

The second is a modification of the hydraulic lift regulator, invented by the late Mr. Appold, used to regulate the descent of hydraulic passenger lifts under a variable load; it has been applied to its new object by Mr. W. Anderson of the firm of Eastons and Anderson, and in some respects resembles the mode of Lieutenant Carroll: the velocity through the pipe of discharge is, however, in this case made to move a suspended plate of curved form, in front of an opening also fixed inside the pipe, and the opening is therefore reduced by increase of velocity.

In December 1866 some experiments were made with a 6-inch Appold regulator at the request of Col. Smith, engineer to the Madras Irrigation Company, and of Mr. Clark, hydraulic engineer to the Municipality of Calcutta.

In one experiment, in which the regulator was used to discharge water from a tank 7′ 7″ square internally during 13 minutes; the surface of the water in the tank sank as follows, in one minute intervals: $3''\frac{5}{16}$, $3\frac{1}{4}$, $3\frac{5}{16}$, $3\frac{1}{4}$, 3, $3\frac{3}{16}$, $3\frac{1}{8}$, 3, $3\frac{1}{16}$, 3, $3\frac{1}{16}$, $3\frac{1}{8}$, $3\frac{3}{8}''$;—the total quantity discharged in 13 minutes was

$$= 7'\,7'' \times 7'\,7'' \times 3'\,5\tfrac{1}{2}'' = 197{\cdot}22 \text{ cubic feet,}$$

or about 15 cubic feet per minute.

In the second experiment, the surface of the water in the tank sank as follows, in one minute intervals: $3''\frac{5}{16}$, $3\frac{5}{16}$, $3\frac{3}{8}$, $3\frac{1}{2}$, $3\frac{5}{16}$, $3\frac{11}{32}$, $3\frac{3}{8}$, $3\frac{3}{8}$, $3\frac{3}{8}$, 3, $3\frac{5}{16}$, $3\frac{1}{4}$, $3\frac{3}{16}$, $3\frac{1}{4}$, $3\frac{3}{8}$, $3\frac{5}{16}$, $3\frac{9}{16}$, $3\frac{11}{16}$, $3\frac{5}{8}$, $3''\frac{5}{8}$; the total quantity discharged in 20 minutes was

$$= 7'\,7'' \times 7'\,7'' \times 5'\,8'' = 323 \text{ cubic feet,}$$

or about 16·13 cubic feet per minute.

In the latter case the heads at the beginning and the end of the discharge over the centre of the pipe were 22·8 feet and 12·24 feet.

In each case the same regulator or module was used; its

square aperture on the delivery side was $5''\frac{17}{32}$ high, and $3''\frac{11}{16}$ broad, or a section of $20''\cdot35$; the swinger was $3''\frac{7}{8}$ wide, nearly touching at top and bottom; the case $5\frac{1}{4}$ wide, and the area for water passage $8\frac{9}{16}'' \times 1\frac{3}{8}'' = 11''\cdot77$ in section.

Two of these Appold's modules are believed to be in use on the Tumbaddra canals of the Madras Irrigation Company. From the convenience of form that this module possesses, being self-contained, and externally a simple iron tube, with an enlargement like a box in the middle of it, that admits of being attached or detached from an orifice very rapidly, it would appear to be far preferable to that of Lieut. Carroll, and less liable to damage in transit.

The third portable self-acting module is the design of the author of this work, and is named the Equilibrium Module. It consists in the first place of a box or chamber, having an entrance and an exit orifice, and one or two air holes above; within this box is the pipe leading from the entrance orifice for a short length horizontally and then turning vertically upwards; this is terminated by a dead end, but has two or four slits or narrow vertical openings in the sides, through which the water passes when the module is open and working. There is at all times enough water within the chamber to rise above the level of these openings, and to work a float above them; this float, working vertically, raises or lowers the cap that slides over the head of the pipe, and gradually opens or closes the slits with the variation of the level of water in the chamber; which must of course be below the low water surface of the canal or tank of supply. The form of construction adopted reduces to a minimum the depth from the water level within the chamber to the openings, which discharge above the sliding collar, and thus causes the loss of head to be unimportant.

This is also a small module, possibly only a quarter larger than the Appold module before mentioned, and equally convenient as regards portability; it is simple in design, being actually little more than one of the old types of equilibrium steam valve applied as a module in a chamber under pressure: it could, however, be made of any size, the adjustment of the sizes of the orifices of entrance, of exit, and of the slit-openings being the only important points of variation. It might also, *for rough purposes*, be made generally of stone-ware,

and the pipe would then be square in section and have only two slits, the other two sides forming part of the box. This module slightly resembles the old cylinder sluice, which is also a modification of a double beat steam valve; the latter, however, is not so simple, being far more liable to choke or get out of order, one of its valves working within the pipe, and it is therefore not so effective in constant use as any of the three already mentioned are likely to be.

2. MODERN IRRIGATION IN ITALY.

THE persistent increase of prices of the necessaries of life in all civilized countries has, during the last half-century, been mitigated by improved communications—the railway and the steamer—with countries less civilized, but more capable of production. That a further and wider extension of such communications will continue to produce a mitigating effect we have little doubt; but afterwards, what have we to look to? Many of the expensive requirements of civilized existence admit of substitutes. For coal we may substitute peat fuel or petroleum; for fabrics hitherto necessary, others less expensive, obtained from plants and grasses hitherto neglected, but now forced by research and skill into the service of man; but, as regards our more urgent wants—bread and meat—there is not now the slightest probability of any substitute being found that could materially relieve the demand for them. We may substitute one kind of meat for another, or one kind of corn for another, as bacon for beef, and maize or millet for wheat and barley: but this is merely economizing by reduction; so we may safely assume that increasing the production of grain and grass throughout the world is the principal mainstay in the future.

In highly civilized countries, where there is comparatively little land fit for culture not already under cultivation, and where high farming has already been adopted to obtain increased produce, it may be assumed that the best results have been nearly reached; it is therefore to less civilized and more distant countries all over the world that we must look for increased produce mainly, and,

in the first instance, by increasing and improving the culturable area.

Of all means of increasing agricultural produce, irrigation stands justly at the head, increasing the yield of the very best lands, rendering inferior lands capable of yielding crops of a superior kind, and apparently nearly useless lands, such as much of the sandy arid plains of India, of yielding good crops of different descriptions; the increased yield obtained by these means supporting men and cattle, and causing, through the manure derived, an additional source of increase. The development, therefore, of irrigation everywhere, its means and methods, its economical application, and the investigation of its results under different conditions, become subjects of interest, not only to the professional hydraulic engineer, but of vital importance and consequent interest to every being existing on the face of the earth. Leaving the history and archæology of irrigation for the consideration of the engineer devoted to such subjects, contemporaneous irrigation has besides a still further interest for the capitalist, everything pointing to the probability that, in and for the future, capital will be largely applied to works of irrigation; the countries where irrigation is likely to be most productive being generally incapable for the present of using capital of their own, and the communications on which capital has been so largely utilized having been so far developed as to set free a large capital for other purposes.

The most interesting irrigation, therefore, will not only be contemporaneous, but that which is most instructive as regards results. The project for the irrigation of a tract of land in Lombardy by the waters of the Lago Maggiore, being carried out in 1872 by a small company of local shareholders, under a concession granted by the Italian Government to its engineers, Eugenio Villoresi and Luis Meraviglia, seems to satisfy these conditions in every respect. The works are not large, it is true; but it does not partake of the nature of an experiment, having an element of stability in it, firstly, from being carried out in a country more or less permanently irrigated since the Middle Ages, and hence instructive as regards the development of principles, and, secondly, from being the result of local effort forcing itself forward, and

succeeding by acting with the wishes of the population, independently of foreign aid.

The comparative smallness of the project, again, has its advantages, in point of interest, from allowing a perfect development within itself, and is thus more truly instructive in showing what might be done on a large scale with large capital, and by the application of the more extended principles not yet adopted in Italy, but already plainly indicated in the large Indian works of irrigation. Some of the details of the scheme and of the intended results will be interesting in comparison with similar data for Spain and India.

The following information with regard to the Lago Maggiore irrigation project, and local matters in connection with it, was obtained during a visit in 1872, from or through the Director of the College of Engineers, the Director of the School of Agriculture, Signor Cantoni, and principally from Signor Villoresi himself.

The tract of land to be watered from the Lago Maggiore is almost entirely in the Milanese province, and is bounded by the Naviglio della Martesana and the branches of the Naviglio Grande; its area is 216 234 acres and its population 459 166. It is peculiarly dry, from causes that have not yet been explained; the inhabitants either collect rain-water, or draw from wells 40 to 100 feet deep, and scanty in the best seasons, or obtain from the pools of the River Olona the water for their domestic wants. The springs or sources of the Olona are now probably less productive than they were, and as its supply is cut off above, for irrigation purposes for an adjoining canal, it is nearly dry in the region under consideration, the eight or nine torrents running into it being of little value. There are also eight torrents running towards the river Lambro, towards the east; but the whole of these, including the springs and the Olona, are not sufficient for the irrigation of 2500 acres of ordinary cultivation according to the usual Italian practice. The tract of land has a generally uniform fall from west to east, and from north to south, of ·75 and ·20 per 100; the soil is alluvial, and classified into four gradations of mixture of sand and clay, covered with a vegetable stratum 7 to 14 feet thick, and occasionally more; the most sandy portions admit of being irri-

gated with good effect, and generally consist of pasture land; on the whole of the rest, however, crops are grown independently of aid, excepting the portions covered with heather and woods, which, from continual cutting, have nearly disappeared. For the crops, the rotation in vogue is biennial; in the first year a first crop of wheat or rye, followed by autumnal maize or millet of some sort, in the second year spring maize. Very small quantities of vegetables, flax, hemp, and ravizzo (colza) are sometimes grown; in some parts of the wheat-growing land trefoil is sown among the wheat in the spring, so as to obtain a first cutting from it in the autumn, and a second in the following spring, but this is very rarely successful for want of sufficient moisture: over a larger portion vines and mulberry trees are planted; in all cases the coloni (rentiers, tenants) paying the proprietor in kind, or taking, in the last case, part of the produce in payment for their labours.

Now, even in its unirrigated state, this is certainly not an unproductive region; there is no mention made of deficiency of crop, and the population is thirteen to an acre, although a certain proportion of the land is scrub, heather, and woodland; and yet the inhabitants have set to work energetically to irrigate and increase the produce. Over how large an area of the world is there not land yielding not one half of this without the slightest efforts being made to introduce irrigation! What millions of acres not yielding a quarter of this, in India, are allowed to remain unirrigated, or, as the contemplative Anglo-Indians in charge would say, uninterfered with!

The introduction of irrigation would, under these as well as almost any circumstances, involve an agronomic change, and a different succession and rotation of crop, to which in this, as in all cases, a certain proportion of the cultivators and proprietors are strongly opposed, although they must, from their close vicinity to other irrigated lands, be fully aware of the advantages of irrigation. It seems difficult to fully account for this feeling so often shown in similar cases. Water has to be paid for no doubt; but there is more produced wherewith to pay. Is it the timidity of entering on matters on a larger scale, and want of self-reliance in adapting themselves to a new system; or is it that unreasoning obstinacy so generally ascribed to agriculturists? Whatever it

may be, the difficulty in this case seems to be partly met by the School of Agriculture, established at Milan, from which more extended ideas on such subjects are disseminated through lectures and ready information within the means of all.

The first agronomic change proposed is the reduction of the whole of the scrub, heather, and woodland into culturable soil; the second, a great reduction of the vine-growing area for the purposes of cultivating corn, the latter change being justified by the fact that the greater part of the wine produced in this region is of very inferior commercial value. The wisdom, however, of this latter change seems open to objection; as a better cultivation of the vine-growing area, combined with winter floodings, could hardly fail to produce a larger amount of wine. Assuming, however, that this change is desirable (and several landed proprietors have adopted it), it will, when general, reduce three-quarters of the vine-bearing area into cultivated or pasture land. The third agronomic change is that of the formerly cultivated land; the biennial rotation having, under irrigation, to give way to a more comprehensive arrangement. A typical rotation has been laid down which is quinquennial, according to the following table:—

Table for Quinquennial Rotation of Crop.

No. of parts.	First Year.		Second Year.		Third Year.		Fourth Year.		Fifth Year.	
	1st crop.	2nd crop.	1st crop.	2nd crop.	1st crop.	2nd crop.	1st crop.	2nd crop.	1st crop.	2nd crop.
2	Permanent pasture throughout.									
1	Colza.	Maize, aut.	Wheat.	Bulato.	Maize.	...	Wheat.	Bulato.	Fallow.	
1	Flax.	Maize, aut.	Wheat.	Bulato.	Fallow.	...	Flax.	Maize, aut.	Rye.	Bulato.
1	Grass.	...	Flax.	Maize, aut.	Rye.	Bulato.	Maize.	...	Wheat.	Bulato.
1	Wheat.	Bulato.	Fallow.	...	Flax.	Maize, aut.	Rye.	Bulato.	Maize.	
1	Rye.	Bulato.	Maize.	...	Colza.	Maize, aut.	Wheat.	Bulato.	Maize.	
1	Wheat.	Bulato.	Maize.	...	Wheat.	Bulato.	Maize.	...	Colza.	Maize, aut.
1	Maize.	...	Colza.	Maize, aut.	Wheat.	Bulato.	Fallow.	...	Flax.	Maize, aut.
1	Maize.	...	Rye.	Bulato.	Maize.	...	Colza.	Maize, aut.	Wheat.	Bulato.
10										

N.B.—The Maize, when not mentioned as autumnal, is spring Maize.

It is drawn up to suit a holder of five acres, with his family and cattle. The quantity of maize produced is one-third more than that from land irrigated on the old system, and is sufficient to support the family. The amount under pasture is as large as can be conveniently arranged, in order to secure as much manure for the soil as possible, and, in the case of a five-acre plot, will support two cows. The wheat and rye grown will pay the rent of the ground to the proprietor, and the spring hay the whole of the irrigation, leaving the remaining crops to the holder entirely free. The same rotation is suitable to a holding of any size, worked by one family, the basis being the proportions of grass, grain, and other crops, which are, taking the whole in ten parts:—two-tenths permanent pasture; one-tenth grass crop; three-tenths wheat and rye; two-tenths to spring maize; two-tenths to autumnal maizes and oil-yielding crops.

It will be noticed that neither rice cultivation nor marcite cultivation—the well-known flooded winter grass crop of Italy—enters at all into the above proposition, being generally excluded from the proposed irrigational demand. This is highly significant, and appears to point to the conclusion that such cultivation is rather on the wane in Italy. Probably it is not economical on well-farmed lands; the winter grass crop is believed to yield only a quarter more through flooded irrigation on the marcitorial system, and both this and rice cultivation are considered injurious to the public health in Lombardy, having been for many years forbidden within certain distances of towns, cities, and villages. In Portugal, lands formerly growing rice are now otherwise cultivated, on economic grounds; experience plainly showing that the production of other grain, and the support of cattle, are more remunerative. In this special instance, as returns are obtained from using the water for motive purposes, driving mills, &c., it is also extremely probable that it is not only more convenient, but also more remunerative, to use water during the winter months in that way.

With regard to the injuriousness of the neighbourhood of rice cultivation, or of any swampy cultivation, there is still considerable doubt. In India rice has been grown close to numerous military cantonments for many years, without any detrimental effects; *whereas, the neighbourhood* of a single rice patch in a fork in the

hills is sometimes almost deadly, and snipe shooting for a few days over rice fields in China and Ceylon is almost certain to cause fever. Medical men have given widely opposed opinions on this subject, as well as on the effects of irrigation generally; from which, apparently, the only sound conclusion seems to be, that irrigation, properly conducted, is perfectly innocuous, and that it is only when the drainage of the country is allowed to stagnate for a long time that injury results. This will perfectly explain how it is that rice cultivation may or may not be injurious, as in some cases the water is allowed to stagnate, unchanged and without flow, for a very long time—a perfectly unnecessary proceeding, which, producing an organic decay more rapid under high temperature, is the cause of noxious miasma. It would hardly seem, however, that in this special case hygienic reasons alone would stop marcite and rice cultivation—as it need not be carried on near villages—but rather reasons of economy. Such a conclusion would, therefore, show that water is more economically expended on other crops, and that irrigation-water is hence becoming more valuable than formerly.

With reference to the amount of water necessary per acre of irrigated land in the tract under consideration, the following four tables supply the data on which it has been based:

TABLE I.

Volume of water in cubic feet necessary per acre at each watering.

	Absorbed.		Utilized.		Expended.	
	Meadow.	Arable.	Meadow.	Arable.	Meadow.	Arable.
De Regis	5 885	8 476	8 665	9 323	14 550	17 799
Cantoni	5 585	8 476	8 411	8 454	14 296	16 930
Committee of Engineers	5 885	8 476	10 404	11 314	16 288	19 790
Totals	...	...	27 480	29 091	45 134	54 519
Mean	5 885	8 476	9 160	9 697	15 045	18 173

Table II.

Quantity of continuous water in cubic feet per second per acre necessary for irrigation.

	Meadow land.		Arable land.	
	For watering once in 7 days.	For watering once in 10 days.	For watering once in 14 days.	For watering once in 20 days.
De Regis	·02404	·01683	·01471	·01029
Cantoni	·02362	·01653	·01398	·00978
Committee of Engineers ...	·02691	·01883	·01635	·01008
Total	·07457	·05219	·04504	·03015
Mean	·02486	·01740	·01501	·01005

Table III.

Area in acres that can be irrigated by one cubic foot per second.

	Meadow land.		Arable land.	
	Watered once in 7 days.	Watered once in 10 days.	Watered once in 14 days.	Watered once in 20 days.
De Regis	41·46	59·22	67·78	96·83
Cantoni	42·19	60·28	71·26	101·80
Committee of Engineers ...	37·07	52 91	60·95	98·86
Total	120·72	172·41	199·99	297·49
Mean	40·23	57·47	66·66	99·16

Table IV.

Supply necessary for each acre of the irrigable area.

	Area in acres.	Sandy soil.		Clayey soil.	
		Quantity of continuous water necessary for one acre in cubic feet per second.	Product.	Quantity of continuous water necessary for one acre in cubic feet per second.	Product.
		cub. ft.	cub. ft.	cub. ft.	cub. ft.
Meadow	1·48	·02486	·03679	·01740	·02575
Arable (?) ...	1·98	·01501	·02972	·01005	·01990
Corn (?)	1·48				
Quantity for ...	4·94	=	·06651	or	·04565
Quantity for ...	1·00	=	·01346	or	·00924

Result adopted for calculation of supply to one acre: in sandy soil, ·01346 cubic feet: in clayey soil, ·00924 cubic feet.

To complete the calculations of this part of the subject, before entering into details and comparisons, it may be said that, dividing the total area under command into two classes, sandy and clayey soils, the total water supply required is as follows:—

			Cub. ft. per sec.	
For	47 674	acres clayey at ·00924 cubic feet =	440	
,,	143 016	,, sandy at ·01346 cubic feet =	1925	
Total	190 690	Total	2365	
		Deducting an already existing supply of ...	310	
		Adding for irrigation in a lower tract of land	388	
		Supply required ... ... =	2443	c.m.
		Hence the actual supply of the canal is fixed =	2825	or 80.

As the whole tract amounts to 216 234 acres, this would show that more than seven-eighths will be irrigated, and, taking the quantities approximately, the average supply over the irrigated area is ·012 cubic feet per second per acre, or is such that 1 cubic foot per second will irrigate 90 acres, from which, according to Table III., the average watering will be once in 18 days or 20 in a year.

Before entering into these general quantities, the principles and details on which they are based require examination.

In table I. the quantity of water sufficient for one irrigation or watering is taken at 15 000 cubic feet for pasture, and 18 170 for arable land; it cannot be doubted, by any one conversant with irrigation in India and Spain, that this quantity is excessive; the Italians of both Piedmont and Lombardy have for a long time been exceedingly wasteful of irrigation water; they have had the unusual advantages of being able to get as much as they like, and as admitted by themselves in Piedmont, the waste is excessive, a natural result of having been provided with too much; in Lombardy again, those that dare to raise their word in private against the traditions of the past have expressed strong opinions that water can there be made to perform a much higher duty than at present.

The object of ordinary irrigation in hot climates is simply to supply the place of rain and soften the soil, and differs much

from the irrigation of lands in colder regions, which, partaking of the nature of sewage irrigation, has for its object the deposition of a fertilizing sediment rather than a supply of moisture, and corresponds in Italy to maricitorial and rice cultivation only. This latter description of irrigation being excluded from the project and data under consideration, the former alone has to be dealt with, and for such purposes in India and in Spain a watering of 10 000 cubic feet is ample, and would doubtless be enough in Italy also, either for pasture or arable land. One such watering represents a depth of ·23 feet over an acre and is equivalent to a continuous supply throughout the year of ·000317 cubic foot per second. It may be said that under different states of climate, soil and subsoil, more water would be required even in hot climates, but to this the reply is that a greater number of waterings might be required, but not a larger supply at each watering. In support of the statement that 10 000 cubic feet would be sufficient, it may be noticed that the learned and scientific Professor Cantoni, Director of the School of Agriculture at Milan, who has been continually and is still prosecuting researches into agronomic and agricultural matters, fixes his quantities lower than the previous data of the older Italian hydraulic engineers, and far lower than those of the Commission of Engineers (about one-eighth less); it is possible also that, were he not an Italian and holding a Government appointment, he might be very much bolder in his reduction.

With regard to the number of waterings, the amount allowed appears, according to Tables II. and IV., to be 52 and 26 in the year for meadow and arable land respectively on sandy soils, and 36 and 18 on clayey soils; but, as the canals are closed for cleansing and repairs during April and October, these numbers are reduced in practical application to 46 and 23 for sandy, and 30 and 15 for clayey lands. Now, leaving out of consideration the fact that these waterings are a half and three-quarters larger than would be requisite in India or Spain, their number seems excessive. In India the number of waterings prescribed on the Nageenah Canal, North-West Provinces, is thus (*vide* "Hydraulic Manual," Part II.): For fruit gardens, 8 per annum; for hemp, 5 per crop; for rice, indigo, sugar, tobacco, grasses, and herbs, 4 per crop; for cotton, wheat, *barley, and grains* and pulses, 3 per crop. In Spain the number

of waterings in the year generally necessary are, according to Mr. Roberts's excellent pamphlet: For corn, flax, potatoes, olives, and vines, 6 waterings; for meadows and artificial grass, 8; and for garden produce, 20; and these by no means show the highest duty obtained by water in Spain, for, in the clayey vegas of Alcanadre and Lodosa, gardens are irrigated with ·0014 cubic foot per second per acre through the year, and only require double or treble that amount, say, ·004 cubic foot per second in very dry seasons; whereas the watering of garden land with twenty irrigations mentioned above requires ·012 cubic foot per second per acre. In both Northern and Southern India ·01 cubic foot per second per acre is considered a full and liberal gross allowance for all crops, except rice and crops grown on the flooding or marcitorial principle, where sediment is the object, while the net allowance per acre yearly appears to be about from one-half to three-quarters of that amount.

The inevitable conclusion, therefore, appears to be that Italian practice gives one-half too much water at each watering, and at least one-half too many waterings, thus employing in detail more than double the water that is necessary according to both Indian and Spanish practice, the conditions of soil and climate being more favourable in Piedmont and Lombardy than probably in either Spain or India.

With reference to the water supply in the gross, or water duty over the whole tract of land, the ultimate duty reached in clayey soil, according to this project, is 110 acres to a cubic foot per second. On previous old works the duty reached in Piedmont and Lombardy seems to vary from 60 to 110, 90 and 100 being the more favourable cases, and 60 to 80 the more usual. In India the water duty arrived at was on the Eastern Jumna Canal, in 1864, 220 acres; on the Western Jumna Canal, in 1863, 280 acres; and on the Ganges Canal, in 1864, 140 acres; and these on canals that are not fully developed, thus pointing to a safe gross water duty, excluding single waterings, of double that obtained in Italy. It is unfortunately useless to mention these things to Italians, whose ideas of hydraulic grandeur and authority are confined to the Naviglio Grande and their old hydraulic authors and engineers; to say to them that there is a canal from the Ganges that is designed to carry a volume of 7000 cubic feet, or 198 cubic metres per second, is

even now unwise, while to attempt to explain that irrigation is not only Oriental in origin, but that ignorant natives of India, led by military men who cannot be called engineers in the civilized Western sense of adepts at scientific construction, but whose proper sphere is the siege and the battle-field, have, in spite of a wonderful chain of mistakes, succeeded in carrying out, not only the largest works of irrigation, but also the most economic distribution, would be intensely absurd.

The increase of produce due to irrigation in the tract under consideration has been calculated by a commission nominated by the College of Engineers of Milan, acting on behalf of the Government of the country granting the concession. Knowing the way in which petty intrigue enters into every matter in Italy, one cannot in this case, any more than in determining the amount of water necessary for the crops, expect unbiassed data. Under similar circumstances, in England, no one would think of curtailing the profits and hampering the undertakings of engineers in this manner; on the contrary, one would think, the greater the profit and freedom, the more likely would be the extension of similar works conducive to the public good as well as to private interest in every way; petty ideas, however, seem to rule in Italy. The data, however, are interesting, and may possibly, after all, be accepted as tolerably correct. The same amount of area has the value of its produce compared under dry and under wet cultivation, and the difference credited as the result of irrigation. The land is divided into four classes according to the degree of sandiness, and the results are given. Those for the extremes of sandiness and clayeyness are alone given in detail; they are as follow:—

TABLE OF PRODUCE.—*Sandy Soil.*

		Produce in Bushels or Cwt. (C)		Price.		Value.						
Dry Land.	acres.	per acre.	product.	s.	d.	£	s.	d.	£	s.	d.	
a. Wheat	1·235	9·023	11·143	4	10	2	12	6				
b. Rye	1·235	12·545	15·493	3	7	2	14	8				
c. Maize	2·471	18·157	44·866	3	0	6	12	0				
d. Maize, quarantine, 2nd crop	1·235	9·023	11·143	3	0	1	12	9				
e. Straw, 2nd crop	1·235	C 5·985	C 7·391	0	9	0	7	7				
f. Mulberry leaves	4·942	C 4·803	C 23·736	1	6	2	8	10	16	8	4	
Deduct for disasters $\frac{1}{9}$th	...	...	...	...		1	16	6				
" drought $\frac{1}{4}$th	...	...	...	...		2	3	0	3	19	6	
Produce of	4·942	...	...	...		...			12	8	10	
Irrigated Land.												
1. Wheat	1·483	9·023	13·381	4	10	3	3	0				
2. Flax	·494	...	...	81	10	2	3	2				
3. Colza	·494	11·775	5·817	7	6	2	2	8				
4. Maize	·988	33·562	33·159	3	0	4	17	8				
5. Maize, autumnal, 2nd crop	·988	25·309	25·005	3	0	3	13	8				
6. Pasture, three cuttings	·988	C 42·052	C 41·547	1	8	4	14	0				
7. Grass	·494	C 42·052	C 20·773	1	8	2	7	0				
8. Straw	1·483	C 11·969	C 17·750	0	7	0	14	8				
9. Erba quartirola	2·965	...	...	9	1	1	8	9				
10. Mulberry leaves	4·942	C 5·591	C 27·631	1	6	2	16	9	28	1	4	
Deduct for disasters generally $\frac{1}{9}$th ...	...	...	...	...		1	17	3				
10. " " $\frac{1}{7}$th ...	...	...	...	...		0	8	1				
6, 7, 9. " " $\frac{1}{15}$th ...	...	...	...	...		0	11	3	2	16	7	Net.
Produce of "	4·942	...	...	...		...			25	4	9	Net.
Increase due to Irrigation	4·942	...	...	...		...			12	15	10	Net.
" per acre	1·	...	...	...		...			2	11	9	

TABLE OF PRODUCE.—*Clayey Soil.*

		Price in Bushels or Cwt. (C).		Price.	Value.		
Dry Land.	acres.	per acre.	product.	s. d.	£ s. d.	£ s. d.	
a. Wheat	2·471	15 076	37·253	4 10	8 15 4	...	
c. Maize	2·471	24·209	59·820	3 0	8 16 0	...	
d. Maize, quarantine, 2nd crop	1·235	12·104	14·948	3 0	2 4 0	...	
e. Straw 2nd crop	1·235	C 5.985	C 7·491	0 9	0 7 7	...	
f. Mulberry leaves	4·942	C 3·543	C 17·510	1 6	1 16 0	21 18 11	Gross.
Deduct for disasters $\frac{1}{9}$th	...	...	...	...	2 8 9	...	
„ drought $\frac{1}{6}$th	...	...	...	...	1 18 0	4 6 9	
Produce of	4·942	...	...	...	...	17 12 2	Net.
Irrigated Land.							
1. Wheat	1·483	15·076	22·358	4 10	5 5 2		
2. Flax	·494	...	...	90 10	2 8 0		
3. Colza	·494	13·425	6·632	7 6	2 8 9		
4. Maize	·988	36·313	35·877	3 0	5 5 7		
5. Maize, autumnal, 2nd crop	·988	26·850	26·528	3 0	3 18 1		
6. Pasture, 3 cuttings	·988	C 42·052	C 41·547	1 8	4 13 11		
7. Grass	·494	C 42·052	C 20·773	1 8	2 6 10		
8. Straw	1·483	C 11·970	C 17·751	0 7	0 14 7		
9. Erba quartirola	2·965	...	...	9 1	1 8 9		
10. Mulberry leaves...	4·942	C 4·173	C 20·623	1 6	2 2 4	30 12 5	Gross.
Deduct for disasters generally $\frac{1}{9}$th ...	...	...	...	...	2 4 6		
10. „ „ $\frac{1}{7}$th ...	...	...	...	...	0 6 0		
6, 7, 9 „ „ $\frac{1}{15}$th ...	...	...	...	...	0 11 4	8 1 10	
Produce of	4·942	...	...	...	...	27 10 7	Net.
Increase due to irrigation	4·942	...	...	...	...	9 18 5	Net.
„ per acre	1·	...	...	...	...	2 0 1	Net.

The gross result from these data is the following increase of value of crop due to irrigation for the four classes of land, namely:

	£	s.	d.	Mean.
1st for the most sandy—per acre	2	11	9	£2 4s. 8d.
2nd ,, ,, do.	2	5	0	
3rd ,, ,, do.	2	2	0	
4th for the most clayey do.	2	0	0	

The results last given seem very small; but it must be noticed that very low values are given to the produce, in all cases only three-fourths the mean local value for the preceding five years being applied; but they are useful in showing how little results vary within the area under consideration. The principal point of importance seems to have been, purposely or otherwise, entirely omitted; the yield per acre of wheat is assumed to be the same both under irrigation and otherwise. The maize, the staple food of the peasantry, is assumed to yield more than a half more, and the mulberry crop of leaves one-seventh more; but the wheat is supposed not to be affected by irrigation. Now in India, where crops are grown dependent on the annual rain, as well as assisted by irrigation, the increase of yield of grain due to irrigation is large; cereals and oil seeds yielding from a quarter to a half more.

There can therefore be little doubt that there must be some increase of yield of wheat also in Italy, and it appears unfortunate that any profits due to irrigation in any way should be allowed to pass unnoticed. If the object is to show as little profit to the landholder, and hence obtain water at as a low rate as possible by similar devices, it is a very shortsighted one, on behalf of the Government of a country, the sole result of depreciating the value and profits of irrigation being to leave the country unirrigated and in an impoverished agricultural state. Taking, however, the figures given by the Commission as relatively correct, these show that the effect of irrigation is to more than double the value of the produce on sandy soil, and to increase it by nearly two-thirds on clayey soil, when the improved rotation of crop is adopted; and at the same time prove clearly that, if allowance be made for an increase in the produce of wheat per acre, and for higher rates for values of produce generally, the value of produce due to irrigation is fully doubled on

the lands that benefit least by it. The importance of so incontrovertible a fact requires no comment, and it becomes a convenient basis for calculating what amount of water-rate could be easily paid under a more correct valuation of produce. Taking, however, the present valuation, which gives as an increase of value per acre £2 5*s.* as a mean, though, perhaps, it would be more correct to assume £3, to determine the water-rate per acre adopted, viz., 10*s.* 9*d.* and 7*s.* 5*d.* for sandy and clayey soils, respectively, it would appear that the water-rate is fixed at a price about one-fifth of the increase of value resulting from the aid of the water, the landed proprietors incurring at their own expense the costs of levelling and fitting their land for irrigation, and keeping their own trenches of distribution in a proper state of maintenance. This is doubtless a very low water-rate; but the circumstances under which this irrigation project is being carried out are peculiar, and the terms of the concession are drawn up to suit the case. But of this more hereafter.

The following is an abstract of the cost of the works of the Lago Maggiore project:

Construction.

Headworks	£259 614
No. 1. Main canal 30 miles, section 604 square feet	215 516
2. ,, 14¾ ,, 320 ,,	126 831
3. ,, 18¼ ,, 341 ,,	102 240
Secondary canals, 132 miles in all	83 659
Keepers' houses	2 800
Legal expenses	15 227
Engineering expenses...	17 684
Interest	50 719
Miscellaneous	5 710
Total ...	£880 000

Maintenance Annually.

Headworks	£1 271
Main canal 63 miles	1 822
Establishment and office	2 400
Imposts	4 507
Total ...	£10 000

Expense per Acre to Landed Proprietors.

	£	s.	d.
Land occupied by trenches	1	7	0
Excavation in trenches		8	0
Buildings	1	0	0
Adapting the land	1	2	0
	3	17	0
Annual maintenance of trenches and administration per acre		1	3

Details.—The headworks, which include a large basin, a large navigation lock for communication with the Ticino, a roadway and sluices, do not seem to have any features worthy of remark. The main canals are constructed to deliver, No. 1, 2825 cubic feet with a fall of ·20 and ·15 per 1000; No. 2, 1766 cubic feet with a fall of ·1 per 1000; besides 209 feet by 26 falls or locks; No. 3, 530 cubic feet with a fall of ·13 per 1000; some portions are paved in boulder work set in earth, or in ordinary lime, and in some cases in hydraulic mortar over a bed of béton, with walled sides. The works on the three main canals consist of:

2 railway and tramway bridges.
7 bridges for provincial roads.
55 smaller road bridges.
13 over crossings for rivers and brooks.
27 locks, mostly with bridges or outlets.
56 falls, syphons, and under passages.
9 keepers' lodges.

The secondary canals are 16 in number, of different lengths and sections; they are generally of four sections.

	Feet.		Cubic feet per second.
No. 1. With a bottom width of	13·1	carrying	103
2. „	9·8	„	78
3. „	6·6	„	54
4. „	3·3	„	30

They have an inclination of ·5 per thousand; and the works consist of 38 bridges and falls for provincial roads, 395 district road bridges, and 397 petty bridges.

The capitalised price of the water in the Lago Maggiore scheme is fixed thus for total amounts:

	£	*s.*	*d.*
Continuous water, per cubic foot per second ...	589	4	7
Summer water	566	11	5
Winter water	22	13	2

Separating this into payments over the forty years in which the project is to repay its costs, and allowing for 6 per cent. it becomes:

	£	*s.*	*d.*
Continuous water, per cubic ft. per second (yearly)	41	7	2
Summer	39	13	2
Winter	1	14	0

And under the agricultural rotation adopted, with the quantity of water necessary for each acre of sandy and clayey land, the price of water per acre is:

	£	*s.*	*d.*		*s.*	*d.*
Sandy, capitalised	7	14	5	yearly	10	8
Clayey, „ „	5	5	11	„ „ „	7	6

Checking the capitalised result per acre as follows:

	£	*s.*	*d.*		
Sandy 143 016 acres at	7	14	5	yields	£1 104 083
Clayey 47 674 „ „	5	5	11	„	252 672
				Total ...	£1 356 755

This shows that the capitalised value of the irrigation effected per acre more than covers the costs of construction and maintenance for 40 years, which is £1 280 000: a further check on this is afforded by capitalising the value of the water per cubic foot. The eventual total supply of the canal will be as before stated, 2825 cubic feet; but as during the first two years the amount to be drawn is, according to the concession, only 1553·9 cubic feet of summer and 1059·5 cubic feet of winter water, from which 5 per cent. has to be deducted for loss by infiltration, the capitalised return would be:

	£	s.	d.		
Summer water 1476 cubic ft. at	566	11	5	=	£836 257
Winter „ 1006 „ „ at	22	13	2	=	22 793
Total ...					£859 050

And the annual return under the same circumstances:

	£	s.	d.		
Summer water 1476 cubic ft. at	39	13	2	=	£58 535
Winter „ 1006 „ „ „	1	14	0	=	1 710
Total ...					£60 245

The returns for navigation are calculated on the basis that the canal will compete successfully with the railway, when carrying goods at half the present railway rates; and, applying this to a boat load of 40 tons sent by either manner, the navigation toll is fixed at 12 shillings per boat load, or about 3½*d.* per ton: it is calculated that such a boat would make 35 voyages in a year going down with the current, but requiring one or two horses to tow it up when empty or full. On these principles the expected return from navigation is estimated at £12,000; by others as follows:

Signor Tatti, engineer	£15 400
Signor Conte Annoni	15 200
The Commission of the College of Engineers of Milan	1 800

The comparison of these data seems, by the evident underrating of the profits of navigation in the last one of them, to throw light

on the over-estimated supply required for irrigation by the Commission of the College of Engineers of Milan, and strengthens the belief before expressed.

The returns for motive power are not estimated, as it is probable that some time may elapse before it is utilized at all; but the amount of motive power is thus calculated: 26 falls on 28 locks having a total fall of 210 feet, having a supply of water, diminishing from 494 to 211 cubic feet, or a mean supply of 353 cubic feet per second, will give 200 horse power at each fall, or 5000 horse power in all on the main canal, and in the same way 1000 or 2000 horse power more on the secondary canals. It will be serviceable for threshing corn, spinning silk, cotton, and flax, paper, cloth, and other manufactures. Other returns may also be obtained from turf and grass-cutting, water for domestic use, wash-houses, and the supply of drinking cisterns for cattle. The total annual return that may be expected amounts to:

By irrigation	£60 245
By navigation	12 000
By other sources	3 755
Total ...	£76 000

Considering, then, the total cost of the works to be	£880 000
And the annual cost of maintenance to be ...	10 000
And deducting this from the annual return of ...	76 000
The remainder	£66 000

represents an interest of 7½ per cent. on the total cost.

It may be interesting, before entering into comment on the subject of cost and return, to deduce the profit per acre that the occupiers of the land can obtain on the whole, assuming the previous data of increase of yield and needful supply of water as the basis of calculation.

The expenses per acre to the landed proprietors capitalised in *the foregoing data* may be reduced to annual payments over the

forty years, allowing for an interest on the capital of 6 per cent., and become thus:

		s.	d.
Land occupied by trenches		1	9
Excavation		0	6
Buildings		1	4
Adapting the land		1	5
Maintenance of the trenches	annual	0	5
Administration of all sorts	annual	0	10
	Total ...	6	3

And the profit per acre is then:

	Cost of water.		Expenses on the land.		Total.		Value of increase of produce.			Profit per acre.		
	s.	d.	s.	d.	s.	d.	£	s.	d.	£	s.	d.
For sandy soil ...	10	8	6	3	16	11	2	8	6	1	11	7
For clayey soil ...	7	5	6	3	13	8	2	0	7	1	6	11
Mean	9	0	6	3	15	3	2	4	6	1	9	3

Besides this profit, the landholder is much benefited by the effect of irrigation, as the labour of ploughing, harrowing and hoeing is much reduced, and again, as so much land is under pasture, his labour there is reduced to nothing; the soil also becomes much improved in time, and the yield again increased beyond the amount calculated: for these advantages the landowner can again demand justly from him an increased rent; and the capitalised value of this increase of rent will be eventually shown in increased saleable value of the land. It is extremely unfortunate that no data are forthcoming on either of these points, especially as there is such a vast extent of land in Northern Italy that has been brought under irrigation at different times which could have well supplied, at least approximately, sufficient information to have given a sound basis on which to rest expected results of this nature. It seems indeed extraordinary that Signor Villoresi, the engineer of the Lago Maggiore project, who has evidently spared no pains in procuring and setting forth so much detail bearing on his scheme, should have neglected to enter into such an important source of return.

To every irrigation undertaking there are three direct and legitimate sources of return.

1. The profit to the shareholders, justly due to them, the capitalists, directors, and engineers, obtained by charging more for the water than it actually costs them, although far less than its value as shown by results.

2. The profit to the landholders or occupiers, whose increase of yield, and hence increase of profit, after paying the water-rate fixed, is due to the supply of water to the land in the first instance.

3. The profit to the landowners by the improvement of their property and land, from the continuous effect of irrigation, and the advantages of having water available.

Besides these, the indirect advantages are innumerable, having their effect on the people and nation generally, as well as on other nations; but these do not admit of calculation: the three direct sources of return, however, do; and it is solely by means of a careful investigation of their results that the true value of the water can be arrived at, with reference to and in proportion to which, and not according to the haggling with the users of the water, a just water-rate can be determined; the success of the irrigation project being principally shown again by a comparison of the cost with the true value of the water.

Failing, therefore, to obtain information on the increase of value of land due to irrigation in Northern Italy, the following data for Spain, from Mr. Roberts's pamphlet, will give some indication of what the increase of value might be:

		Dry per acre.	Irrigated per acre.
Rioja district	rent	9*s*. to 12*s*.	£9·5 to £10·2.
Zamora, Castile	value	£14 to £18.	£35 to £41.
Near Madrid, 1st class land	value	£32	£128
,, ,, 2nd ,,	,,	£20	£100
,, ,, 3rd ,,	,,	£12	£72
,, ,, 4th ,,	,,	£6	£60
Ampurdan, Cataluna	,,	£100	£200 to £300
Spain generally, 1st class land	inc. of value		100 to 200 p. c.
,, ,, *inferior land*	,,		1000 to 1500 p. c.

This would indicate that it is most probable that the value of the land in Northern Italy would be at least doubled by irrigation.

In India, again, where canals have, at least in a very incomplete and rough state, existed for many years, the profits to the land-owner are very plainly shown. A large portion, if not all, of the immense tracts watered by the Ganges Canal and the Eastern and Western Jumna Canals are, like most of the land in Oriental countries, the actual property of the Crown or government of the country; and the rent of the land in these tracts is newly fixed after certain periods—five, seven, or eleven years—the enhancement of rent on the land as it becomes brought under irrigation being determined at those intervals and credited to the effects of irrigation, as well as the water-rate paid by the occupiers of the soil. Turning for figures to "Hydraulic Manual," Part. II., we find, among the returns given by canals :

		By water rate. £	By enhancement of rent. £	Total returns. £
Eastern Jumna Canal in	1846	12 175	14 965	27 140
Western Jumna Canal	1845	29 888	37 000	66 888
Ganges Canal	1867	136 353	80 018	216 371
Ganges Canal	1868	244 156	161 260	405 416

These show that in two out of the three great canals the enhancement of rent is a larger source of return than the water-rate, and that it is only on the least developed canal of the three that it is less, and even then amounts to two-thirds of it. This is, no doubt, under what we should call in European countries an exceptional state of affairs; and it is evident that under such circumstances, where the owners of the works and the water are also the owners of the land, they could, if they preferred it, give the water gratis, and raise the whole of the returns by means of enhancement of land rent. In Southern India, unfortunately, the practice prevails, of throwing into one payment the water-rate and the land rent, so that one is unable to distinguish between the two species of returns.

On the whole, however, these figures, as well as those for Spain, show incontestably that the landowner makes an immense

profit from the results of irrigation; whereas the water owner has to haggle over a petty water-rate with the occupier, in order to make it possible to carry out the works at all; or, in other words, every one profits highly from the water except those through whose skill and management the water is supplied; and, more, it seems likely that this state of things will continue until the immense profits of irrigation are fully set forth in such a way that ignorance of them can no longer be professed. When this is done, a more adequate water-rate can be demanded, and will be cheerfully paid by the occupier, and a second water-rate should also be demanded from the landowner.

The necessity as well as the justice of a second water-rate from the landlord has been very recently shown in India, although, of course, much opposition was made. There are certain districts in India where the soil has been either alienated from the Crown by gift at some period, or has been put under a permanent settlement of land rent in perpetuity, that cannot be enhanced. In undertaking works of irrigation in such districts, the Government saw itself deprived of the main source of return obtained in other cases, through the right of landownership, and the difficulty was, therefore, met by an Act of the Imperial Government at Calcutta in 1870, drawn up by General Strachey, Inspector-General of Irrigation, which, among other matters connected with the subject, ordained that a water-rate should be paid both by landlord and occupier, and besides, that a certain small water-rate should be paid by those owning or holding land within an irrigable area, but declining to use the water or sell their land. This Act marks an era in irrigational matters, and points the way to the rest of the world by which carrying out irrigation projects may be rendered, as they should be, sufficiently remunerative to those that undertake them; much praise, therefore, is due to the then Inspector of Irrigation for carrying out such a measure, which must have originally met with great opposition in a country like India, where the natives will haggle over giving a halfpenny or a penny for every pound one may put into their pockets, and where the English, having generally falsely so-called liberal notions, would, in most instances, not understand the *justice and true* liberality of such measures.

These principles and facts may be said to have established for the future, in Europe and elsewhere, that a second water-rate can be justly demanded from the landlord after the land is fairly brought under irrigation, or, if alternatively, that the occupier's water-rate can be increased, so as to include the two rates in one payment, leaving him to settle his own proportion of it with the landowner.

Having thus pointed out how important an element of profit has been neglected in the calculations of the Lago Maggiore project, having indicated its value, and shown how it might have been raised, let us return to the consideration of the cost and return data.

First, as to the works themselves and their design, there seems to be very little deserving of special comment: from the remarks made in the drawing up of the project, the use of hydraulic cement is treated as a novelty in Italy, and is mentioned as having been adopted in France and Belgium; this may be considered as the key-note of the indigenous engineering. For measurement of water, the old *modulo magistrale* of Milan, with the *oncia magistrale* as the unit of water measurement, remains in its pristine state; sluices and outlets are also very primitive. Imagine ourselves in England at the present time using the unimproved locomotive of Trevithick and the cast-iron rails of that period, and we can understand the progress of the Italians in a branch of engineering equally important to them as improved communication is to us; while, therefore, criticism, on Italian engineering construction is quite out of place at present: this objection, however, does not extend to examination of the results of their works of irrigation.

The complete scheme, costing £880 000, will irrigate 190 690 acres with 2472 cubic feet per second, out of the 2825 of full supply; allowing proportionately for the remaining 353 cubic feet per second, we obtain, as the grand result of the whole scheme when in working order, an irrigation of 217 930 acres, or a tract of 389 square miles, allowing one-eighth as unirrigable, for the total cost—that is, the cost per irrigable acre is a little more than £4, and the cost per square mile of irrigable tract is £2256. Referring to Roberts for similar information for Spain, we get:

Cost per acre.

		£	s.
Province of	Madrid	15	6
„	Logroño	7	15
„	Toledo	6	5
„	Gerona	8	6
„	Leon	7	10
„	Navarra	7	0
„	Guadalajara	6	10

These are the results of carefully compiled estimates, that allow for all contingencies as well as for liberal contractors' profits, of which there is no mention in the data of the Italian project.

For India we obtain the following results on partly developed canals.

	Total outlay.	Irrigated area.	Tract irrigable.
	£	Acres.	Sq. Miles.
Ganges Canal, 1864	2 058 714	449 788	
„ „ 1870	2 402 438	...	16,000
Eastern Jumna Canal, 1846 ...	81 460	421 875	497
Western Jumna Canal, 1846 ...	119 405	351 501	1 345
Rohilcund Canals, 1864 ...	31 190	83 904	—

These show results of 3, 4, and 5 acres irrigated for £1; and, allowing for difference in cost of labour to the very utmost amount, one acre would be irrigated in India for £1, against £4 in Italy, or £5 or £6 in Spain.

The differences in cost may perhaps be accounted for in the case of Spain, by supposing that the estimates for the works there are for really good construction in the English style. In the case of India, it may be remarked that the acreage there given is the sum of the acres irrigated, continuously, in the autumn, and in the spring; *e.g.*, the total yearly irrigation or acreage of the Ganges Canal for the year 1868, given as 1 078 400 acres, is composed of 60 664 acres continuous, 293 604 spring, and 734 132 autumn irrigation; this must then be carefully borne in mind with reference to Indian irrigated areas; but even after making allowance for this, the Indian construction seems by *far more economical* in prime cost.

With regard to maintenance, the annual cost in the Lago Maggiore project is £10 000 for 217 930 acres; for Spain there are no available data, but in India we have:—

	Estab. and Repairs.	Acres.
Eastern Jumna Canal, 1846	£7 340	421 875
Western Jumna Canal, 1846	12 584	351 401
Ganges Canal, 1868 ...	75 731	1 078 400

and the comparison indicates that, after making sufficient allowance for difference of cost of establishment and labour, the former costing far less and the latter far more in Italy, that maintenance costs more in Italy.

The expense per acre to the proprietors in preparing the land, trenches, &c., is £3 17*s.* per acre in Italy, against £6 for corn land, and £12 for garden land in Spain; but this is a matter that depends so much on local circumstances, that the comparison is of little value. Nor again is the point of expense to the occupier of great importance. In most land fit for irrigation the expense cannot be very heavy; the work is done by the occupier and his family or field hands during the time that would otherwise be unoccupied, or at least comparatively so; and the labour expended is more than counterbalanced in perpetuity by the saving of work in the operations of ploughing, harrowing, and hoeing on irrigated ground, which is considerably less than in dry land.

The data of cost of all sorts, taken with reference to the acreage, do not thus indicate any advantages in point of economy in favour of Northern Italy over Spain and India; it has not, of course, been possible to obtain strictly corresponding data, but it has been shown quite possible to draw undeniably just comparisons from those given, after making due allowances.

It may be urged that, to relieve the comparisons entirely from any doubt regarding acreage, it would have been better to keep them entirely in terms of cost, price, value, and return per cubic foot per second of supply; of this there is little doubt, and it would have been so arranged had sufficient data been available in that form; there are, unfortunately, none forthcoming for Spain, and as for Indian returns, in many cases, the terms, cost, price, and value are used without proper discrimination: taking these returns

rigidly, the cost should be the expense of the works, or capital account, per cubic foot supplied; the value should represent the whole of the benefits valued and summed per cubic foot, and the price the simple water-rates paid. Such data, however, as can be procured are as follows:—

	Supply.	Total cost.	Cost per cubic foot per second.	Price yearly per cubic feet per second.
	cub. ft.	£	£	£
Lago Maggiore project ...	2825	880 000	307	42
Ganges Canal, 1870	4300	2 402 438	558	44
Eastern Jumna Canal, 1870	956	194 575	206	62
Western Jumna Canal, 1846	2800	119 405	42	24

These are not very instructive, as the supply mentioned is probably not in all cases the supply actually utilised in irrigation alone, and the price yearly may in one case not include, as it should, the amounts obtained by increase of land assessment. It must be remembered, also, that the data for Lago Maggiore are those of a completely developed project, whereas in the Indian data they are, excepting the last, those of only partly or imperfectly developed works.

That the water-rate of the Lago Maggiore project, 10*s*. 8*d*. for sandy, and 7*s*. 5*d*. for clayey soils per acre, is very low indeed, may be shown by comparison with the following rates in Spain, most of which are fixed merely to pay for repairs and guards, the works belonging to the land without having any interest to pay off.

		Water-rate per acre yearly.
Canal del Urgel	...	19*s*. 3*d*.
Tagus Valley ...	...	10 per cent. of the produce.
Malaga ...	...	19*s*.
Lobrigat ...	...	5*s*. 6*d*. to 17*s*.
Aragon ...	...	4*s*. to £1 7*s*.
Cataluna ...	...	12*s*. to 16*s*.
Navarra ...	...	12*s*. for four irrigations yearly.
New Canals ...	...	1*s*. 7*d*. to 2*s*. 4*d*. for each watering.
Frequent custom	...	10 per cent. of the produce.

If 10 per cent. of the produce determined the water-rate on the Lago Maggiore tract, it would be from £2 16*s*. to £3 1*s*., instead of *7s. 5d. to 10s. 8d.*, and this would probably be a fairer water-rate.

But enough has been put forward to show how the project has been plundered of its apparent profits, by requiring too much water per acre, and, besides, by underrating points on which the water-rate and the estimate of the value of the results of the water have been based.

This happens to be of no importance whatever in this special case, as the association carrying out the works consists of users of the water, occupiers and landowners, who make and take the whole of the profits in whatever shape they may appear; their object is to clear themselves of the prime cost of the works in 40 years, and retain the works permanently as their own after that time; and as they can do so by so fixing the rates as to pay only 7½ per cent. on the cost, this arrangement suits their purposes. Beyond this, the conclusions one would be liable to arrive at with reference to this scheme, that 7*s.* 6*d.* or 10*s.* 8*d.*, are just and fair water-rates for Northern Italy, and 7½ per cent. is a fair profit on such works there, are evidently false.

If these works had been carried out by shareholders not owning or holding the land, a really remunerative water-rate of as much as a half of the value of the increase of produce resulting from irrigation, which is evidently much more than £2 4*s.* 8*d.* per acre, could be easily paid by the occupiers in the first instance, still leaving large profits both to occupiers and landowners, and from the latter again the second water-rate might be demanded; the works themselves might also be sold at some fixed price either to the Government or the landowners for a hundred years, having easily paid 30 per cent., as the preceding examination has shown. When it is considered that, even then the landholders would be enabled to increase permanently the value of the produce of their lands by one-half without any risk or investment, it seems extraordinary that Italian landholders have not already largely invited the use of foreign capital for such undertakings and hypothecated their lands with this object.

The preceding inquiry into the value of the results of irrigation will, it is hoped, have furnished ample evidence of the immense profits to all concerned that works of irrigation can produce, and demonstrated clearly that it is solely due to a want of careful investigation that they have been so much ignored hitherto.

3.—THE CONTROL OF FLOODS.

The prevention of the submergence of land by inundations from overcharged rivers, and the drainage from marshes and submerged land of the water that has been allowed to accumulate over it, are kindred engineering problems that appear at first sight to present but little difficulty. Their theoretical solution, when merely on a small scale, is ready and simple; on a larger one, however, the practical details brought into these problems affect them to such a degree, that, although the principles involved cannot be said to be subverted, their carrying out is forced into a comparatively new form.

Land liable to submergence from a river is lower than the extreme flood level, and in open communication with it; the remedies consist, therefore, either in lowering the extreme flood level in the channel by providing other passages for the water, partially diverting it, or dredging out a deeper channel, or by warping up the land liable to submergence, or by cutting off possible communication in flood stages between the river and the land by means of embankments. Submerged land, again, remains in that condition for want of sufficient natural outfall; an outfall has, therefore, to be cut, tunnelled, dredged, or enlarged to a sufficient extent to allow gravity alone to do the work, should that be possible or economically sufficient; in other cases pumps are indispensable.

Imagining, then, the case to be one of an area of a few hundred acres, liable to inundation from a river with a moderate declivity, the application of these principles involves generally but little difficulty as regards engineering, and becomes a local economic question, rather than an engineering practical problem. Putting the case again on a large scale, a vast tract submerged by the floods of a river having a very small declivity—the usual condition when large areas are submerged—the dimensions entering into the works that would be necessary in adhering rigidly to the above principles become so large, that their complete execution is positively impossible in most cases. Let us adduce the embankments of the

Ganges, the Mahanuddi, the Po, and the levées of the Mississippi, which are not and never can be complete and sufficiently developed to insure, by means of themselves alone, the absolute protection of all the lands on their banks from the devastating effects of extreme floods.

To this it might be, though perhaps rather thoughtlessly, replied, that very extensive works may be so costly as to be impossible, but that the application of the principles need not vary. It is, however, in point of fact also a matter of modification of the application of principle.

The case of a comparatively small river supplying the flood, very nearly, and in most cases totally, limits the consideration of the flood to its principal point, the extreme flood level; the catchment area of a small river being tolerably uniformly supplied throughout the rainfall, its upper portions do not require very special consideration; the declivity of the small river being tolerably rapid, the condition of the lower ranges of the river does not affect the matter to any very important degree. Remote local conditions being comparatively disregarded, and it being possible to cope with the flood at the required point both successfully and economically, the works involved are necessarily small.

On a large scale, on the contrary, the extreme flood level, the nature, causes, and duration of the flood may be greatly affected by any of the physical conditions of the entire catchment area of the region watered by the river and its tributaries, from the loftiest hill on the watershed down to the currents of the ocean, miles beyond the river's mouth; and as these physical and meteorological conditions vary greatly throughout large countries, a perfect knowledge of them as regards the country under consideration is absolutely necessary in order to arrive at sufficient information to enable one to propose measures for the mitigation of the effects of the flood. In other words, the natural drainage of the whole region under any state or circumstances, as well as everything that practically affects it in any way, must be thoroughly known in detail.

It will be unnecessary to dilate on the physical laws and conditions of our sphere, matters best understood from studying the larger works on physical geography to be found in any good

library: and a knowledge of these will hence be assumed. The detailed knowledge, however, of the physical conditions, and specially of the rainfall of the region under consideration, may possibly not be obtainable from any book whatever. It is not sufficient to possess meteorological statistics of observations taken at a few towns in the valley of the river, and at one or two points or villages on the hills; it is needful to know definitely what is the greatest amount of rain that ever falls in the region, the greatest area in it over which rain falls at any one time, and which portions of the area they are likely to be at any time; or generally how much water, when, and where, so that it may be practically accounted for. Detailed observations taken for many years at a very large number of meteorological stations are therefore requisite, and it is almost painful to reflect in how very few instances are even a moderately small number forthcoming. As a notable exception to this apparent apathy, may be noticed the large number of meteorological stations in the United States of America, and the large sum annually spent by their Government in obtaining such information. Besides the meteorological data, a correct detailed topographical and hydrographical knowledge of the whole of the catchment of the river, based on engineering surveys and velocity observations, is necessary in order to determine the discharge and the flood level of the river at any time, and under any possible meteorological condition. Having all this information, we are enabled at any time to state what will be the results in rise and amount of discharge of the river, corresponding to and resulting from any special rainfall lasting for any usual or unusual time over an area, or detached portions of area within the catchment basin, and the evils to be contended with are then fully known before commencing to deal with them and attempting to mitigate their ill effects by means of engineering works of any sort.

To this it may be replied, that the expense of obtaining all these data, and especially those of a hydrographical and topographical nature, which cannot be done except by skilled hydraulic engineers, must necessarily be very large; and if after all this it should be discovered that under any circumstances no engineering works could remove the evils, or even moderate them to an *important extent,* the expense would have been uselessly incurred.

Not entirely so. Even should no works be attempted, the information can be made use of in the protection of human life, and in thus mitigating the fearful effects produced by sudden and devastating floods. The extent of land liable to submergence under certain conditions of rainfall in any part of the country being known to a practical certainty, the telegraph can be employed to warn the inhabitants of an impending flood, and allow them to save at least their own lives, and perhaps also that of their cattle and movable valuables. It may be urged that the terrible catastrophes resulting in large loss of life generally commence with the bursting of an embankment, which happens before the flood overtops it; doubtless it is so, but it would be an important part of the topographical knowledge to ascertain to what height of flood these embankments, which, when in sound condition, are in most cases only sufficient protection against very moderate floods, are practically safe. Timely warning could, therefore, be afforded in any case, and the inhabitants would be spared the terrible infliction, in case of flood, of watching the waters rising, and not knowing either how much higher they might rise, or to what height of flood their dams might be safe.

But to proceed to the main object, the protection of the land, as well as its inhabitants, when the matter is one of large extent and importance.

The usual practice hitherto, notably in the case of several districts in Holland, seems to have been, to construct continuous lines of embankment along all the existing edges of the various channels of the river, and discharge the waters within them on the flooded land into the rivers by means of pumps. This caused no doubt a certain amount of mitigation of evil up to certain height of flood level only; beyond that, it is sufficiently evident in theory, and has been fully established in practice, that the means employed cease to be a remedy, and become a decided aggravation of the cause of disaster, effecting an excess of external pressure on the embankments. Besides this, as the channels of the river are under these circumstances allowed to silt themselves up, not only the bed level, but also the flood level corresponding to the same amount of discharge, is allowed to rise also; a second aggravation of the evil. Beyond this again, the immense length of these circuitous em-

bankments causes them to be exceedingly costly. These three reasons will, it is hoped, have sufficiently demonstrated the fallacy of employing the means, that are occasionally appropriate on smaller works, to those of large extent.

Before entering into the subject of works based on better principles, let us first examine the conditions of a flood under circumstances that admit of easy personal observation.

Let us imagine ourselves to be standing on the bank of an Indian river, as wide as the Thames at Hammersmith, in a mansun season of unusually high rainfall, the maximum annual rainfall being 74 inches, the day maximum 7 inches. The mansun, or periodic rainy season, has set in tolerably mildly; the river swells, increases in depth and velocity, and is discoloured at first; this afterwards passes away, and the water then runs steadily, tolerably clear. The rain increases in the plains, and the sky gives prospects of a heavy storm in the direction of the uplands of the river. Let us watch the effect. The rainfall of the plains, in fact the downpour all around us, increases the depth and the velocity of the river, but its colour is unchanged, in fact it seems nearly pure. Suddenly a roaring of waters, like that below an overtopped mill weir, is heard, and up stream we notice a white line of foam approaching; three or four minutes, and a flood sweeps by on the surface of the river, like a wall of water 3 or 4 feet in height; all this water is muddy and dark with detritus. The waters after this again rise still higher for twenty-four hours, but are yet muddy; the low-lying lands near the river are submerged. We learn afterwards that a considerable fall of rain has taken place in the uplands of the river, and that towns and villages in the plains have been inundated.

Such is the flood, its subsidence is a matter of less moment; and such is the type of flood to which those causing serious catastrophes generally belong. In this case we fully satisfy ourselves of the rationale of the flood; the lowland water rises steadily and clear, going perhaps one mile an hour; the upland water comes down with a velocity of nearly six miles an hour and charged with silt,—for where else is this velocity and this silt to come from *except* from its course in the hills?—and tops the lowland water; *the combination of* waters gradually decreasing in speed spread

themselves out over the land in the first locality, where the form of channel and banks admit of it, and perhaps in more than one, extending even for miles beyond the natural bed of the river.

How is such a flood to be controlled? Apart from the Dutch principle, already shown to be fallacious on a large scale, there are only two methods, either or both of which can be adopted. The first, the improvement of the whole of the natural drainage lines of the country to such an extent that the velocity of the waters may under such circumstances be increased throughout the whole course of the river, and a little beyond it, into the sea or next large river, and so that the natural bed, thus improved, may be sufficiently large to carry off any previously known flood, without being exceeded. The second, any means of separating the upland from the lowland waters, holding or retarding either the one or the other, or portions of either one or the other, and providing for their discharge either separately in different courses, or at different times in the same watercourse. Let us first indicate the nature of the works requiring execution, when the former principle alone is adopted: the perfecting of the natural lines of drainage.

The ultimate free delivery of the water into the sea, or any way entirely free of the river, is perhaps the most important point of all, the low-lying lands on the lower ranges of the river being there more extensive than elsewhere; to insure a free delivery, the main outlet of the river should be carried out to deep water, protected on both sides by banks or jetties, against the shore currents, and so directed as to avoid as much as possible the retarding influence of sea storms; through the delta, also, a single direct channel of properly determined dimensions should be made and protected by embankments; by these means the mass of water will, in forcing its way in this course to the sea, scour for itself a deeper bed at the outfall and throughout the lower ranges of the river, and carry off floods more rapidly, improving the river continually. A further advantage from confining the river to one channel is that of the reclamation of a large amount of land previously occupied by marshes, as well as by the numerous old channels of the delta.

In the middle ranges of the river the works to be adopted are all such as will promote a more rapid discharge: the enlargement of the bed wherever it is contracted or narrowed; the removal of

obstacles, rocks, small islands, silt deposits, shoals, or anything that impedes velocity; the straightening of the course wherever it can be done to good effect; the prevention of the deposit of silt in such places as would be objectionable; the deepening or dredging of the bed in the requisite places: the whole course to be put under a regimen that would remain constant generally, and besides continue to improve itself by scouring in contradistinction to its former habits of silting up and causing its flood levels to rise.

In the uplands, all the works which should be constructed are those that have for their object the control of the detritus washed down, and the prevention of its deposit at unfavourable spots. If the silt could by any means be entirely prevented from being carried down into the middle ranges of the river, or into the plains, it would be a great achievement; but this being hardly possible, palliative measures are perhaps all that can be adopted. Besides this, the hills might be covered with thick plantations, which, catching the rainfall, would delay its departure, prolong the duration of the flood, and thus lessen the amount of flood water passing off at any one time, or mitigate the flood.

The necessary works dependent on the second of the principles previously mentioned, would be so greatly dependent on local circumstances that they can only be indicated generally. The separation and control of the water from the uplands can be attained by making storage reservoirs at certain places at the foot of the hills, and running all the water falling on them into these by means of catchwater drains skirting the bases of the hills; from these reservoirs the water can be allowed to escape under control into the main watercourse; or, if practicable, the upland waters may be discharged through very large catchwater drains, independently of any reservoir, into some other collateral watercourse that may be convenient, employing even, if necessary, a separate outlet for the discharge into the sea of the upland waters.

In the case, however, of the main river or watercourse being employed as the outlet for the upland waters, it becomes necessary to separate the lowland waters from them as long as possible. In order to do this, the arterial drainage lines of the plains on each side of the main river require rectifying and improving; their waters then have to be cut off from it, and carried by two canals

down the valley of the main river as far as some point where it may be advisable to discharge them into it through regulating sluices, or, if preferable, into some artificial reservoirs or lakes. These latter works would insure the additional advantages of perfecting the entire drainage of the country, and of having a good water supply for irrigation.

The adoption of the two principles thus described would insure a perfect remedy and an effective control of floods under any practicable circumstances. That such works would necessarily be expensive there is no doubt whatever, but they would still be less costly and more effective than the continuous lines of embankment designed on the fallacious principles before quoted; the works again would improve the rivers instead of deteriorating with lapse of time, and the gain by reclamation and irrigation would, apart from other collateral advantages, yield a profitable return.

4.—TOWAGE.

Recent experiments show that the pull on the towrope of a barge is, within practical limits, proportional to the square of the speed, and that it varies widely according to the form of the barge; assuming then a general formula,

$$R = b\,T\,V^2$$

where R is the resistance in lbs.,

T = the displacement of the barge in tons,

V = the velocity through the water in miles per hour,

and b is a coefficient depending on the form of the barge.

It has been found that for the small and bluff barges of about 70 tons employed on the Thames, and for limits of speed not exceeding 5 miles an hour, the coefficient $b = \frac{1{\cdot}5}{\sqrt[3]{T}}$ or generally about 0·369; and that for well-formed barges of medium size,

$$b = \frac{0{\cdot}75 \text{ to } 1{\cdot}00}{\sqrt[3]{T}} \text{ or generally about } 0{\cdot}170\,;$$

and for the best ship-shaped barges with good lines, as those

employed on the Danube wire-rope system, which have a length about eight times their beam, and are about 287 tons' displacement,

$$b = \frac{0\cdot5 \text{ to } 0\cdot6}{\sqrt[3]{T}} \text{ or generally about } 0\cdot109.$$

The limit of speed for ships will be about 10 miles an hour, and beyond these limits the resistance R would vary with the fourth power of V: but within the assumed limits, calculations may be made on the above data.

The number of horses required to draw a train of barges may hence be readily deduced. The best performance of a draught horse working 8 hours per day, is assumed to be at the speed of $2\frac{1}{2}$ miles per hour, when he will exert an average pull of about 120 lbs.; substituting this value in the above formula, we obtain for the tonnage that one horse will pull at the speed of 2·5 miles an hour in still water,

$$T = \frac{R^1}{0\cdot17\ V^2} = \frac{120}{0\cdot17\ (2\cdot5)^2} = 113 \text{ tons.}$$

In a current, the resistance or the pull upon the tow-line will increase as the square of the speed through the water, but the horse in this instance moving over the ground is going at a less speed than that of the boat through the water; and this is an important distinction, which must not be overlooked in estimating the effect of a current. The mode in which the necessary correction must be effected will be best illustrated by an example.

Referring to the last example, let us assume that the barge of 113 tons' displacement encounters an adverse current of 1 mile an hour, and it is required to know the reduced speed at which the horse will then go, assuming him to be performing the same average work per hour.

In the last case, the said work in mile-pounds was $120 \times 2\cdot5 = 300$ mile-pounds per hour; in the present case the pull upon the rope will be proportional to the square of the velocity, through the water (V), and the pull the horse is capable of pulling will be inversely proportional to the velocity at *which he is travelling* (v); and the difference between these

two velocities will be the speed of the current (v_1): we have therefore

$$V = v + v_1 \text{ where } v_1 = 1 \text{ mile per hour}$$
$$R = \cdot 17\, T V^2$$
$$\text{and } R v = 300 \text{ mile-pounds per hour}$$
$$V^2 (V + v_1) = 15{\cdot}4$$

whence $R = 19{\cdot}4\, V^2$, and $V^3 - V^2 = 15{\cdot}4$.

Solving which we obtain $V = 2{\cdot}86$ miles per hour, the speed of the boat through the water;—and the speed past land, or rate at which the horse is going, will be $2{\cdot}86 - 1 = 1{\cdot}86$ miles an hour.

It will be observed from this example that the influence of the current is relatively less important when horses are employed, than when steam-tugs, either paddle or screw, are used, the reason being that in the latter case the reaction operates upon the moving current, whilst in the first case against the immovable tow-path. Thus, in the present example, if the horse instead of being an animal moving on the tow-path had been a steam horse in a tug, the speed through the water would be the same, whether the water was still, or ever so rapid a current. In this instance 2·5 miles an hour the speed past the land, which is the useful result, would be reduced to 1·5 miles an hour in the case of the tug, instead of to 1·86 when horses are used.

The difference of conditions will be more strongly marked if we assume the current to be 2·5 miles an hour, because then it is obvious that the steam tug, capable of moving through still water at that rate, would simply maintain its position if it encountered such a current; and although the paddle-wheels or screw would be revolving at the same rate as before, the only result of their effects, namely, the maintenance of position of the boat would be equally attained if she dropped anchor; in short, the whole power exerted would be thrown away. In the instance of the barge towed by horses, on the other hand, the whole power exerted would be utilized; and it may be shown by the same reasoning as in the last example, that the 113 ton barge would be towed by one horse against a current of 2·5 miles an hour, at the rate of $1\frac{1}{6}$ miles an hour.

Obviously the same reasoning would apply, whether the motive power on the tow-path were horses or a locomotive, or whether

the tow-path were dispensed with, and a rope were laid down in the bed of the river, and coiled round a drum in a steam-barge in the manner now generally admitted to be the most economical mode of conducting heavy traffic at a slow speed in rivers of rapid current and on still-water canals.

From the above we may conclude that, in order to tabulate for the effect of a current on the diminution or increase of speed of a horse, we have to calculate the increased or diminished value of V the velocity through the water, and apply it in the general formula—

$$R = bTV^2$$

inserting different values for the constant b, which lie between ·109 and ·369, according to the form of the barge.

In the above case R = 120 lbs. for a draught horse; but for other animals corresponding values of R, with reference to their best continuous speed, can be applied.

Assuming a case of a current of 3 miles an hour, and that the ordinary limits for the speed of the horse in towing a load with and against stream, are 4 and 1 mile an hour respectively, the velocity through the water becomes 1 and 4 miles an hour, and the loads 706 and 44 tons, the horse performing the same average work, but executing the average pull of 75 lbs. with stream, and 300 against it.

The values required are given for the limits in the following form.

For barges having 113 tons' displacement, and a coefficient $b = 0{\cdot}17$, the results are as follows:—

With the current.			In still water.	Against the current.		
$v_1 = 3{\cdot}0$	2·5	1·0	0	1·0	2·5	3·0
$V = 1{\cdot}79$	1·88	2·2	2·5	2·86	3·66	3·97
$v_2 = 4{\cdot}79$	4·38	3·2	2·5	1·86	1·16	·97
$V_3 =$	5·00	3·5	2·5	1·5	0	—0·5

Here v_1 is the velocity of the current, whether favourable or adverse.

V is the velocity of the barge through the water.

v_2 is the speed of the horse.

V_3 is the velocity through the water for the case in which a steam-barge is used, and is given to illustrate the comparison.

5.—ON VARIOUS HYDRODYNAMIC FORMULÆ.

The results of the various formulæ given for determining discharges, according to various authors, vary very greatly; and it is hence interesting to examine them in a tabulated form in comparison with measured discharges.

The following data of comparison are given by Mr. David Stevenson, and by Captains Humphreys and Abbot; they apply to four cases of river discharge, from a small stream up to the Mississippi; thus including all limits within which such formulæ are required.

1. For a small stream of 24 cubic feet per second. Mr. David Stevenson made careful measurements, and velocity observations, and compared the measured results with the results of formulæ, thus:

1. Measured discharge	24·22
2. By Dubuat's formula	32·50
3. By Robinson's formula	36·90
4. By Ellet's formula	46·40
5. By Beardmore's tables	38·92
6. By Downing's formula, coefficient 1·00	41·23
7. By Leslie's formula, coefficient ·68	28·04

2. For a river of 2424 cubic feet per second. Mr. David Stevenson and Dr. Anderson made velocity observations on the Tay, at Perth, and the comparisons are thus:

1. Measured discharge	2423
2. By Dubuat's formula	2987

3. By Robinson's formula 2560
4. By Ellet's formula 2083
5. By Beardmore's tabular formula 2609
6. By Downing's formula, coefficient, 1·00 2769
7. By Leslie's formula, coefficient, ·68 2083

It is unfortunate that in these two cases the hydraulic data, which would enable us to extend the comparison to other formulæ, are not given.

3. For a large river of 31 864 cubic feet per second; the data of the Great Nevka, measured by Mr. Destrem, were as follows:

Area of section	15 554	width	881
Discharge	31 864	perimeter	893
Mean velocity	2·0486	maximum depth	21
Slope . .	·000 014 87		

The following are the results due to these data calculated by various formulæ for mean velocity of discharge:

1. Measured discharge 31 864
2. Young's coefficient 21 102
3. Eytelwein's coefficient 23 389
4. Downing's coefficient... ... ... 25 031
5. Dubuat's formula 16 931
6. Girard's formula 22 491
7. De Prony's canal formula 22 357
8. Young's formula 19 777
9. Dupuit's formula 23 546
10. St. Venant's formula 21 811
11. Ellet's formula 13 807
12. Mississippi new formula 39 938

4. For a very large river, the Mississippi at Carrolton, the measured data at high water in 1851, were,

Area of section	193 968	width	2653
Discharge	1 149 948	perimeter	2693
Mean velocity	5·9288	maximum depth	136
Slope	·000 020 51;		

and the corresponding results, which are kept in terms of mean velocity to reduce figures, were,

1.	Measured	5·9288
2.	Young's coefficient	3·2400
3.	Eytelwein's coefficient	3·5898
4.	Downing's coefficient	3·8434
5.	Dubuat's formula	2·7468
6.	Girard's formula	4·8148
7.	De Prony's canal formula	3·7271
8.	Young's formula	3·2741
9.	Dupuit's formula	4·8752
10.	St. Venant's formula	3·4907
11.	Ellet's formula	3·0451
12.	Mississippi new formula	5·8903

A careful examination of these results in four cases of rivers cannot fail to be instructive; but before entering into comment on the discrepancies and their peculiarities, let us also examine the following list of total discrepancies of mean velocity in thirty cases of rivers, streams, and canals of all sizes given by Captains Humphreys and Abbot in the Mississippi Report, which would, no doubt, be more instructive were the cases classified as to size.

The total discrepancies are:

1.	Measured mean velocity of discharge discrepancy ...	0·
2.	Young's coefficient	32·9420
3.	Eytelwein's coefficient	28·4411
4.	Downing's coefficient	26·6988
5.	Dubuat's formula	40·4417
6.	Girard's formula	37·4472
7.	De Prony's canal formula	28·0905
8.	Young's formula	33·3834
9.	Dupuit's formula	25·1488
10.	St. Venant's formula	30·6619
11.	Ellet's formula	45·3547
12.	Mississippi new formula	6·3920

From this last table of dcrepancies it appears that the Missis-

sippi new formula is by far the most correct, and after it the formulæ of Dupuit and Downing, while the two worst are the formulæ of Ellet and Dubuat; but then it must be remembered that the greater number of these thirty cases are those of large and very large rivers.

In the fourth of the previous cases, a very large river the Mississippi new formula is by far the most correct, and then come in order of correctness, Dupuit, Girard, and Downing, while Ellet and Dubuat are again the worst. In the third case, Downing is most correct, then Dupuit, afterwards the Mississippi new formula, Ellet and Dubuat again the worst. In the second case Ellet and Dubuat remain the worst, and the best are Robinson, Beardmore, and Downing. In the first case Leslie and Dubuat are best, and Downing worst.

It will be understood that the formula mentioned as Downing's being more familar to many under that name, is really that of d'Aubuisson, but applied to English measures.

The inevitable conclusion from all these comparisons is that not one of these formulæ is correctly applicable to rivers of different sizes, nor holds its own equally as regards correctness throughout. For the few and special cases in which the discharge of an extremely large river is required, the Mississippi new formula would necessarily be used, in spite of its form being rather unwieldy; and in the same way Dupuit's formula for a large river. But for ordinary general purposes the thing that the practical hydraulic engineer requires is a formula tolerably well suited to all cases and of a simple form, so as to admit of easy rapid calculation. The most simple type of formula is that of Downing or d'Aubuisson, which gives for mean velocity of discharge

$$V = 100\,(RS)^{\frac{1}{2}}$$

where R = mean hydraulic radius

and S = mean hydraulic slope;

and this, too, is the formula shown to have been generally the most correct throughout all the comparisons and discrepancies, failing only in the very smallest streams, and evidently worse according as the stream or discharge is less; this then is evidently the best formula for general purposes, and simply requires modification by experimental coefficients to answer all ordinary requirements.

The formulæ of Young, Eytelwein, Beardmore, Stevenson, and Leslie, all belong to this type, merely using other numerical coefficients instead of 100.

Putting Downing's formula into the general form

$$V = c \times 100 \, (R\,S)^{\frac{1}{2}}$$

where $c = 1$ according to Downing,

the values of c, according to the other formulæ of the same type are thus:

	$c =$
Young, for large streams	·843
Neville, rivers, velocity $<$ 1·5 feet	·923
" " " $>$ 1·5 feet	·933
Eytelwein, generally	·934
Beardmore, open channels	·942
Stevenson, for rivers of 30 cubic feet ...	·690
" " 2500 cubic feet ...	·960
Leslie, small streams	·688
" large streams...	1·
Downing, Taylor, D'Aubuisson } for open channels	1·

From the comparison of the results of the formulæ containing these coefficients, we may then tabulate values of c that will be practically correct, when suitably applied into the general formula. The comparisons before mentioned show that Downing's coefficient 1·00 gives too small results in cases when the area exceeds 7000 square feet, with a mean velocity of 2·5 ft., or a discharge of 17 500 cubic feet per second, and too large results for cases of smaller data; that the Eytelwein coefficient ·934 in the same way is too small above and too large below discharges of about 2000 cubic feet per second; and the Young coefficient ·843 is incorrect for everything above 900 cubic feet per second; also that for petty streams of 25 cubic feet per second, a coefficient of about ·600 is tolerably correct.

It is evident then that with a very large number of cases of carefully measured discharge, this principle of determining practical coefficients in relation to approximate volume or velocity might be carried out to further exactness; allowances for irregularities,

lateral bends, and so forth, being made independently of this coefficient, as would be done in any case.

Some tabulated values of c, determined in this way, suitable to canals in earth in good order, are given in Chapter I., page 30.

To apply this same principle to discharges through pipes, taking the same general formula,

$$V = c \times 100\,(R\,S)^{\frac{1}{2}}.$$

As this formula becomes more convenient in practice in terms of the diameter of the pipe (d), it becomes for full cylindrical pipes and tubes, where $R = \frac{d}{4}$

$$V = c \times 50\,(d\,S)^{\frac{1}{2}}.$$

And again as the actual discharge is the quantity most often wanted, this is

$$Q = A v = c \times \cdot 7854\, d^2 \times 50\,(S\,d)^{\frac{1}{2}}.$$
$$= c \times 39\cdot 27\,(S\,d^5)^{\frac{1}{2}};$$

and transposing this

$$d = \frac{1}{c^{\frac{2}{5}}}\,\cdot 23\left(\frac{Q^2}{S}\right)^{\frac{1}{5}}.$$

Taking, then, an example in order to compare the results of the various formulæ,

Let Q = 18·57 cubic feet per second
S = 1 in 1276,

and the results then are for diameter:

1. By Dubuat's formula	33·74
2. By Neville coefficient ·228	36·80
3. By the above formula, coefficient 0·23	37·12
4. Young's modification of Eytelwein	37·17
5. Beardmore, coefficient ·235	37·92
6. Hawksley (in Box's tables)	39·59
7. De Prony and Darcy	47·71
8. De Prony's modification of Dubuat	48·16
9. Gerney	48·84

Besides these, there are very many authors that would give results for diameter very much below that of Young; it appears also *that* none of these formulæ apply equally well to both high and low *velocities of discharge*, although it is unfortunate that a sufficiently

large number of data are not forthcoming to determine correctly the limits at which it would be advisable to change the coefficient.

The above comparisons while showing the merits of the various formulæ in certain cases, also point to the very evident conclusion, that a variable coefficient of discharge is necessary both for rivers, channels, and pipes; and that it must be suitable both to the dimensions and the conditions of each particular case. The best mode now known of doing this is that of Mr. Kutter of Bern, which is applied to English measures in Chapter I. of this Manual: the values of coefficients being also given in the Working Tables.

6.—IRRIGATION FROM WELLS IN INDIA.

There is unfortunately a large number of Indian officials that believe that irrigation from wells in India is more profitable than irrigation from canals conveying the water of rivers and delivering it on the surface of the land by the aid of gravity alone; they generally are men not likely to be persuaded to the contrary by engineers, however good their reasoning might be; and, unfortunately, engineers are not always provided with facts and figures. To these latter, therefore, the following data may be of service; they were drawn up as applicable to the years 1855 and 1870; the former by Captain Baird Smith, the latter by the author.

Comparison of Irrigation by Wells with that by Canals for a District of 1 500 000 *acres in Northern India in* 1855.

By wells—Capital :—		
Wells costing £20 each, for 10 acres	...	£3 000 000
Machinery, etc. (and bullocks ?) ...	...	1 000 000
		£4 000 000
Annual expenses :—		
300 000 men at £3 a year ...	...	£900 000
1 200 000 bullocks at £1$\frac{1}{5}$ a year	...	1 440 000
10 per cent. interest on capital ...	...	400 000
		£2 740 000

By canals :—

(assuming the data of the Eastern Jumna Canal).

Capital :—	
1 500 000 acres at 5*s*. an acre	£375 000
Annual expenses :—	
Water rent at 2*s*. an acre	£150 000
Watercourse repairs at 7*d*. an acre ...	43 750
Labour at £2 8*s*. per annum	72 000
10 per cent. interest on capital	37 500
	£203 250

Comparison in favour of Canals.

Capital 1 to 9; annual expenses 1 to 13.
Saving effected annually 2½ millions.

Comparison of Irrigation by Wells with that by Canals in Northern India, for 1870.

Data.—The Eastern Jumna Canals in 1864–65 had cost 15*s*. per acre irrigated; the Western Jumna, up to 1863–64, had cost 12*s*.: hence assuming 20*s*. for a less favourable canal.

	Per Acre. £	*s*.	*d*.
By canal :—			
Capital expended on a developed canal should not exceed	1	0	0
Return levied by water-rate, dues, and increase of land assessment	0	15	0
Working expenses	0	5	0
Net profit 50 per cent.	0	10	0
By wells :—			
Capital expended on a well 10 feet deep with machinery, &c., to irrigate 10 acres at a cost of £30, gives a cost per acre	3	0	0
Working expenses, including interest on prime *cost*	1	0	0

Comparison in favour of Canals.

Capital 1 to 3; annual expenses 1 to 4.

Saving effected annually on a district of 1 500 000 acres, $1\frac{1}{8}$ millions.

The profit of 50 per cent. net, allowed in the last comparison, has already been exceeded on the Eastern Jumna Canals; nor is that nearly so high a profit as might have been effected had the works been carried out steadily, continuously, and by experienced engineers, under arrangements that would have caused or forced the landholders at once to utilize all the water, or sell their lands to others that would do so.

Other important data in connection with irrigation canals are, the saving effected by doing away with remission of land assessment in famine years, and the value of the produce and cattle saved in years of drought; the indirect advantages to the country and the Government, resulting from increase of produce and of population, are innumerable. Well irrigation, on the contrary, fails at the time when it is most wanted, the ordinary wells, being shallow, drying up in years of drought.

In the Hydraulic Statistics are some data having reference to irrigation from wells in different parts of India.

7.—THE WATERING OF LAND.

The following is the usual mode of classifying crops with regard to their special treatment under irrigation. 1. Grass meadows, or natural meadows of gramineæ. 2. Dry grain crops or cereals. 3. Leguminous crops. 4. Root crops. 5. Those specially requiring more water: rice, indigo, tobacco, sugar, bamboo, water-nuts. 6. Garden or fruit crops. 7. New plantations, and trees.

Peculiarities of climate, soil, and water will generally affect the amount of water required for irrigation probably more than than the species of crop. In England meadows of grass land, or Italian rye-grass, are those that generally profit most from irrigation. The usual plan is to keep the land flooded to a depth of

two inches during the months of October, November, December and January, for twenty days at a time, and then to let the water drain off from it for five days, before putting it again under water. In frosty weather, however, the field should always remain flooded. In February and March the fields are flooded for eight days at a time at night only; at the end of March the land is left dry; and in May the grass-crop is cut. Irrigating fields in England in the hot weather is liable to produce rot in sheep, but does not harm cattle.

There are two methods of laying out the courses or channels in English fields:

1. The bedwork system, applicable to flat land.
2. The catchwater system, applicable to steeper country.

According to the former, the land is made into a series of very flat ridges, having a general direction nearly at right angles to the channel of supply, and being never more than 70 yards long and about 40 feet wide, the inclination of the ridge itself having a fall of about 1 in 500, and the inclinations of the sides of the flat ridges varying with the retentive power of the soil, from 1 in 100 to 1 in 1000; the crown of the ridges is not necessarily, therefore, in the middle of the breadth of the base of the ridge. The feeding and drainage channels are generally from 20 inches wide at their junctions to 12 inches at their ends.

The catchwater system used in Devonshire and Somersetshire consists of a series of ridges made across the general course of the water, which hold the water up, and retain it over successive long strips, the water passing slowly round the end of one ridge to the lower land above the next ridge, and so on. This is necessarily far cheaper than the other system—about half, and can be carried out at the cost of about five pounds an acre.

Throughout the world generally, there may be said to be only four methods of distributing water on or throughout surfaces, of which all others are mere modifications. In all cases it is best that the land should have one general slope throughout, the irrigation channel running along the head of this slope, the main catchment drain along the bottom.

The first method is that to which the English bedwork system

belongs, the field being prepared in furrows and ridges alternately from the head to the foot of the slope, either in the direction of the fall or making an angle with it, according as the quality of the soil and the general slope of the land may require; these flat furrows, being from 10 feet to 50 feet wide and only a few inches in depth, receive the water from the irrigating channel, which will then cover the land nearly up to the crests of the ridges, or in fact entirely if need be.

The second method is very similar to the first, but the water, instead of flowing in the furrows, runs in little trenches cut along the crests of the ridges, overflows the sides, waters the slopes, and drains off in the furrows down to the main catchment drain. The ridges used in this system are generally wider than those of the first system, and have a greater lateral inclination.

The third or commonest method for applying water on a small scale is to distribute the water in little trenches around small squares and rectangles of land, allowing it to permeate throughout the surface enclosed, which must be very nearly level with the water in the trenches.

The fourth method, most commonly adopted in Spain, Portugal, and India, in cases where much water is required to remain on the land for some time, as for rice-crops, or many grain and other crops in their early stages, that could not thrive on hard baked soil, consists in levelling the land into a number of nearly flat squares and rectangles, divided from each other by small ridges or dwarf mud walls, to hold the water on them. The number of rectangles depends on the fall of the ground; the water is allowed to flow in at some corner or temporary break, and flow out in the same way on to the next rectangle when it has remained sufficiently long.

As to soil:—For the surface, the most permeable is best, being most easily warmed, and allowing the water to arrive at the roots of the grass most quickly; a retentive surface-soil causes evaporation, and cools the land, which is generally a disadvantage, though not so under some circumstances;—a subsoil of clay, being retentive, is an advantage in very dry climates, as it economises water. In hot climates the soil is of inferior importance to the quality of the silt transported and deposited.

As to the quantity of water required for irrigating a certain area:—In Piedmont and Lombardy one cubic foot per second waters 50 to 100 acres of marcite or grass-land, or only 40 acres of rice; in England the amount required is generally also 1 cubic foot per second per 50 to 100 acres; in the Madras Presidency and in the North-West Provinces 1 cubic foot per second waters in ordinary seasons 100 acres of rice, or other very wet cultivation, but in very dry seasons the duty is as low as 50 acres; but taking all the crops watered throughout, counting single waterings and all the duty per cubic foot per second, is 200 acres both in Northern and in Central India;—the hightest duty actually performed being about 270. In Northern India one cubic foot per second waters 4½ to 5½ acres for 24 hours. But details as to amount necessary in Spain, Italy, France, for Orissa, the Panjab, and India generally, will be found in the Hydraulic Statistics.

As to quality:—Pure water is bad for rice cultivation, and is always far inferior to that which brings fertilizing particles with it. The best water for irrigating land may be said to be that which brings with it a fertilizing matter most suitable to the improvement of the land under irrigation. As a rule, water containing much hydrous oxide of iron is very bad; so also the water that comes from forest or peat-moss is inferior. The water that comes from a granite formation, holding potash, is good; so also is water that comes from pure carbonate of lime; if the water is brackish, it is no objection; salt water meadows are highly productive. A good method of foretelling the effects of the water is by observing the natural products of the irrigating water, such as the grasses and plants that grow on its borders.

With regard to the temperature of the water, very cold spring water is not generally good, and crops require careful preservation from the effects of frost in winter. Warmed water is generally advantageous, and causes rapid growth; it is probably for this reason that water that has been long exposed to air, soil, and sun is more fertilizing than it was in its previous condition. Morning and evening are the best times for watering. The long exposure of the water is much affected by the inclination of the land; the inclination of the main channels in Lombardy is about 1 in 3600, in *Piedmont 1 in* 1600, in Provence 1 in 1000, in Tyrol 1 in 500 to

1 in 300, in Northern India it is generally kept between 1 in 1000 and 1 in 2000. In India generally it is usual so to arrange the inclinations that the resulting mean velocity of current may never exceed three feet per second.

In connection with the watering of the land, the management of its drainage is a matter of the highest consequence. Modes and styles of drainage are necessarily varied, according to local circumstances; but they all have one main object, to keep the circulation of the water and the air through the soil under perfect command, so that the periods of intermittency may be so managed as to suit the soil, the crop, and the circumstances. Any want of good management on this point, is liable to cause most deplorable results; stagnation, causing decomposition and malarious effects in the neighbourhood, and even, in the case of sewage irrigation, making the very crops grown to be useless as food for man or beast.

For the healthy support of crops, a certain amount of water and of stimulant may be used advantageously (*see* Hydraulic Statistics: Watering of Crops in France); beyond this, any addition is worse than a loss—it is a positive source of injury—clogging the soil, and preventing it from fulfilling its necessary functions. With regard to the period of intermission advisable, it probably varies greatly; recent experience in England would, however, seem to show that equal intervals of watering, and of draining off, for twelve hours at a time, afford the most rapid way of utilizing in irrigation as much sewage as possible: further experience, however, is perhaps likely to show that this is not by any means a rule to be followed generally in all soils and conditions.

Assessment of Water-rate.—There are three principles on which water-rate may be levied on land.

1. By fixed outlet, or by measurement.

The small channel of supply being constantly full and of a certain section, the rate may be charged at so much per square inch or square foot of section, independently of the amount of pressure, for a certain time, as by the hour or day of 24 hours. This has been adopted in Italy, but has not been found to act well.

A further development of this method is to measure by module

all the water as distributed, a mode more likely to be adopted at present, now that modules are less expensive and more effective than formerly.

2. By area of land irrigated, or by crop.

This has the following disadvantages; the land to be irrigated is always varying in amount, and this cannot be watched in detail continually, nor can the landowners be trusted to state truthfully the amount of acreage over which water has been distributed. The crop can also be varied, so as to use more or less water, and the payment by crop also would be useless against cheating. Again, in a good rainy season the cultivator might try under these circumstances to do without the canal water, thus causing the water-rate to be precarious.

3. Water distribution by rotation.

An irrigating channel of fixed dimension, giving a constant fixed discharge, passes through the lands of several proprietors; a period of rotation is fixed for this channel, from 6 to 16 days according to the crops, the former for rice and the latter for meadow land, as, for instance, in Italy. Each landowner can then have the whole volume of the channel turned on to his land once in the total period of rotation for a certain number of hours, as from two to forty or fifty according to the amount of land he owns.

For example. Let ten days be the period of rotation, and let him require twelve hours' supply once in that period. His name is placed on the list, say sixth, and he gets his supply turned on at a fixed hour and turned off at a fixed hour also. If the channel gives twenty cubic feet per second, his amount of water is equivalent to a continuous discharge of $\frac{20 \times 12}{240} = 1$ cubic per second. In this way intermittent supplies admit of mutual comparison.

Last with regard to the cultivators themselves:—Whether on the Continent, or in England, the farmer is generally a grumbler under any state of affairs. In India the cultivator invariably complains, although his assessment is very small by comparison with the local circumstances; if he grow two very moderately good crops in the year, it would only amount to about two and a half per cent. per annum on the value of the produce, and he can therefore well afford to pay large water-rates, especially since both the yield and

the number of crops produced on irrigated land is doubled, and the highest water-rate is small in comparison with the expense of making wells and raising the same amount of water by animal power throughout the year; he enjoys also the advantage of living under a government that remits the land assessment, and distributes food gratis in years of famine, while not demanding more assessment in years of plenty. If the water-rate is in some just proportion to the increase of produce and saving of expense resulting from the irrigation, it matters not how large per acre the rate may appear to be. If the irrigation is applied to suitable land in such a way that the natural drainage of the country is not interfered with, there can be no detriment to the health of the cultivator; this can, however, be rarely carried to perfection in actual fact. To this it can be replied, that the population will thrive on the whole and increase largely, which may be considered as a set-off on that account, and that landowners who prefer going away can always do so and part with their land at a premium; land always commanding a ready sale. A compulsory water-rate on land that is under water command cannot be considered a hardship by any one that considers the subject in a fair, unprejudiced manner; the privilege of being able to obtain water should be paid for, and since the same principle has always been applied to town supply of water, for which every inhabitant has to pay whether he uses it or not, there is no reason for leaving the payments of water-rate in the country to be optional. Whether both the landowner and the occupier should pay separately for the advantages they both receive is a point dependent on the local tenure of land; under ordinary circumstances they doubtless should do so, the occupier being benefited by increase of produce, the landowner by increase of rent; but in any case the whole of the advantages should be paid for.

8.—CANAL FALLS.

That a fall of water at the headworks, or at any part of a canal, should be allowed to remain unutilized, appears, in these days of expensive fuel and costly motive power, to be a very painful waste of a valuable advantage. One's natural tendency is to devise means and ways of using everything, and to imagine

that there could hardly exist circumstances under which it would be necessary to arrange for the destruction of the power and velocity generated by a fall of water. Grinding corn, pressing sugar, or extracting oil, are requirements even in semi-barbarous countries, by which such motive power could be easily utilized, even if it were available for only four months in the year. In spite of this, however, it seems rather frequently to occur, that in distant countries the engineer has to devise means for destroying the effect of a fall of water; this occurs, generally, either at the headworks of a canal, where the water entering the canal in flood seasons has a great head of pressure, or at certain points in a canal where, owing to the inclination of the country being steeper than that due to a convenient velocity of canal current, it has been found necessary to concentrate the super-abundant fall: the Ganges Canal and the Bari Doab Canals have many such examples. In either case, as the fall is independent of navigation of any sort, which has to be conducted in a special channel of détour, the problem is one of economy. The natural means would be to break up the force of the water by both lateral and vertical breaks and angular obstacles, and to oppose the remains of the velocity by a pierced breakwater, beyond which the water would issue with so small a current as not to be able to cause any damage to the bed and sides of the canal, or to cause any prejudicial effect to navigation.

The breakwater, involving an enlargement of the width of the channel, and, if a rock foundation be not available, requiring artificial and carefully made foundations carried to some depth, is necessarily expensive, and is hence generally dispensed with, except under favourable circumstances.

The fall itself is generally a modification of one of the three following types:—

1. A uniform, or a broken general incline.
2. A vertical fall with gratings.
3. A vertical fall with a water-cushion.

The most primitive mode of managing such falls of water was to conduct it down an incline, made as gradual as possible, and break up the velocity by a series of steps.

A long reach of rocky bed offers a convenient opportunity for

such a construction, which could be hewn in the solid rock. In other cases, where it would require building on artificial foundations, the expense would be very great; and, even if the incline were so made that the resulting velocity were not high, the edges of the treads of the steps, even in good stonework, would soon wear, and the maintenance of the fall would also become an important item of expense. Apart from these objections also, this type is unsatisfactory. Although the treads of the steps may be set with a correct reverse inclination, so as to oppose more directly the inclined direction of motion of the momentum of the water; and, although a further improvement may be made in giving a more considerable reverse inclination to the treads, and by allowing a large proportion of the water to run off laterally and wind down the steps; yet under all circumstances the inherent defects remain; the steps cannot accommodate themselves to the variation of the quantity of water passing down the fall; if the steps are small, they fail to receive effectively the over-falling water when the amount increases, and become then comparatively valueless; if the steps are very large, the rise and tread of each step causes the velocity acquired from each step, which it must be remembered increases in the ratio of the square of the height of the step, to be very much increased, and to become very destructive to the stonework.

The next improvement on the inclined type of fall is the ogival fall used on the canals of Northern India; in this the general slope of descent from the head to the foot of the double curve is from one to six to one in nine; the upper one-third of the slope being the chord of the upper or convex curve, which is tangential to the surface of the water in the upper reach; and the lower two-thirds of the slope being the chord of the concave curve, which is tangential to the convex curve above, and tangential to the horizontal line at its lower extremity. The height and length of the fall applicable to any special case is determined by equating the discharge of the open channel above with the discharge over a weir. The principle which this form of construction asserts is that the water at the foot of the descent, being deprived of all vertical action and delivered horizontally, will not cause any damage to the bed of the channel in the lower reach.

In canals where it is required that the discharge should remain perfectly uniform and unaffected by its fall down the weir or incline, these ogival falls must necessarily have their sills raised above the level of the channel-bed of the upper reach; as would also a fall of uniform slope.

Curves on more carefully eliminated principles have also been tried with the object of effecting some improvement, but the advantages resulting appear comparatively small. These curves generally effect no doubt some saving of masonry in comparison with that for a single uniform slope, and probably deliver the water with less destructive result than the latter; they are, however, still expensive, and the action of the water delivered is rather concentrated, and hence destructive. An attempt at economy on such falls has been made by narrowing the fall, and thus diminishing the amount of masonry; but the results, caused by the increase of action as well as irregularity of effect of the water, require greater expenditure in repair; they present also the additional disadvantage, that during repair the whole fall instead of a part has to be stopped.

In the above cases of inclined falls it is supposed that it has been found convenient to concentrate the fall in a comparatively short length; in other cases, where it is spread over a long reach, it is usual to attempt to annihilate the velocity resulting at the foot of the incline by introducing a reach of canal having a reverse slope; and in cases where a greater length still can be allowed for the incline, to break it up into portions of descent, each followed by a portion with a reverse slope and then a short horizontal length, thus opposing the accelerating effect in detail without allowing its results to accumulate. In such work the bed of the channel must necessarily be paved; if the velocity do not exceed 10 feet or 12 feet per second, large rough convex boulders, laid dry, form the most suitable paving; and even up to 15 feet per second the same method may be adopted if very large boulders alone are used; beyond that velocity the boulder work requires packing with shingle and pebbles, and grouting with good hydraulic mortar.

While the above arrangements may destroy a great deal of the velocity, there is perhaps almost always a certain amount of it still *remaining at the foot* of the incline, and should the channel at this

place happen to be in soft soil, further arrangements, tail-walls, brushwood spurs, or piles, are also necessary.

The Bari Doab Canal tail-walls offer an example illustrating such a case, the arrangement being generally as follows: At the foot of the incline the bed of the channel is made horizontal for some distance, and the banks are then splayed outwards in a curved form until the top width of the channel at water level is one-half wider than before: this, giving additional water way, reduces the velocity; the channel is then narrowed to nearly its normal width by walls of dry boulders on each side, which project into the stream at an inclination of 1 to 5, and slope longitudinally with a fall of 1 in 20 from their commencement, where their height is up to full supply-level, down to the level of the bed: these are, of course, totally submerged at full supply, and produce the effect of concentrating and directing the current to the middle of the channel. The objections raised to these tail-walls as employed on the Bari Doab Canal, is that they do not appear to answer their purposes sufficiently completely, and it is supposed that by giving the whole arrangement, both the enlargement and the reduction of section, a greater length, it would fully answer all purposes; this, however, would add greatly to the expense.

Vertical falls with gratings.—This is, perhaps, the most economic and convenient mode of dealing with a canal-fall. The sill of the fall is not raised above the bed of the upper channel and the whole section of passage is hence unimpeded by reduction; the grating, which may be placed at any slope from 1 in 3 to 1 in 10, presents a large perforated surface to the action of the water, thus keeping the upper water up to its proper level, and distributing the effect of the falling water passing through it on a long portion of the bed, diminishes the action to such an extent as to render it harmless. The gratings are supported on cross bearers, which again rest on masonry piers or iron stanchions, erected at about 10 feet intervals along the edge of the fall or weir. The higher a fall of this description is, the more truly the water falls and the more manageable it is. These gratings require clearing occasionally, and hence necessitate the attendance of a man; but as frequently there is a lockman to attend to the neighbouring lock, for the navigation passage near the fall, there is

no additional expense incurred on this account, as one man can attend to both. This type of fall admits of comparatively little variation in design.

Vertical falls with water-cushions.—This is the form generally adopted by nature in discharging water down a fall; the action of the water scours for itself a basin, which fills and forms a natural water-cushion, the scour continuing until an equilibrium is established between the force of the descending water and the resistance offered by the depth of water in the basin. The fall itself has a tendency to approximate to the vertical, the force of wind and spray from the falling water making it slightly overhanging, and in some cases even causing a retrogression of fall, and coincidently also a retrogression of water-cushion, thus giving it an elongated form; the scoured silt, or débris, is deposited in the bed of the stream lower down.

The most natural mode of designing a vertical fall with water-cushion for a canal would perhaps depend on a consideration of what sort of fall nature would make for herself under the special circumstances and conditions of the case, and what improvements or modifications of that would be necessary. The objections to allowing nature to make her own fall and water-cushion are these:—first, it requires time, and this, in some, though not in all cases, is an objection in itself; second, any want of homogeneity of the soil or rock would result in an irregular form of basin, which might become almost unmanageable; third, the scour and silt deposited in the channel below would be a serious injury to it; fourthly, the retrogression of the fall might eventually undermine the weir or dam, and cause its entire destruction. But this latter objection might be very easily counteracted by protective measures.

In cases, then, where these four objections can be removed or are unimportant in result, there is no reason why a natural or a slightly modified natural fall should not be adopted. When the soil is firm or of homogeneous rock, a great deal of the objection disappears, a certain amount of excavation and trimming can then be so made as to aid in the natural action, and lateral encroachment may be easily provided against; a tolerably *regular* basin can then be economically made.

As to the form of basin best suited for a water-cushion, the breadth in plan should be rather wider than the extreme breadth of the falling water, as the wind may bear the latter considerably to one side; the length, again, will probably vary from 1½ to 5 times the breadth, although it would hardly be advisable to make it quite rectangular in form, as the corners would be filled with useless water; the pear shape, therefore, is perhaps the best, and is certainly that most generally met with under natural conditions of homogeneity of soil. There would probably be no advantage, even if it were economic, to make the basin very long; the full or extreme depth may be terminated by a reverse slope at once, the deflected velocity thus obtained producing a greater degree of stillness than the passive effect of a longer continued full depth.

The main point, however, is to determine what depth of water is necessary in a water-cushion. The velocity of delivery is evidently dependent on the depth on the weir sill or fall above, and the height of fall down to the surface water in the basin: the resistance is the depth of water in the basin, and the quality of the material of which its bottom is composed. If, then, the depth be calculated by equating the forces for a depth producing equilibrium just clear of the bottom, we obtain an expression, involving also an assumption that the bottom is perfectly indestructible. It seems, therefore, impossible at present to determine absolutely the actual depth necessary; and hence the practice is to assume an approximate calculated depth, and see how this answers its purpose, altering or adding afterwards until it appears to be satisfactory.

The formula generally used for this purpose on the canals of Northern India is—

$$d = 1{\cdot}5\ \sqrt{h_1} \times \sqrt[3]{h_2}$$

d = the depth of water in the basin;
h_1 = the total height of fall, including h_2;
h_2 = the depth or head on the weir sill.

This is probably very limited in its range of application; for, in applying it to the well-known case of the projected Masur reservoir dam, designed by the engineers of the Madras Irrigation Company, it yields results very small in comparison to that allowed by the engineers: thus, for values of $h_1 = 43{\cdot}5$ and $h_2 = 6$ feet, the

calculated value of d, suitable to a brick bottom, is about 18 feet, while the engineers have allowed for a hard rock bottom a depth of water-cushion of 33 feet in this instance.

In a second instance of the same case, the formula gives for values of $h_1 = 16{\cdot}81$, $h_2 = 8{\cdot}56$, $d = 12{\cdot}54$, which is very much less than that allowed, 16·19 feet, also in hard rock.

Major Mullins, the Consulting Engineer to the Madras Irrigation Company, when commenting on these cases in the Proceedings of the P. W. D., for April, 1868, refers also to a well-known natural fall as an illustration of the insufficiency of the above formula. The Rajah Fall at Gairsappa, with values of $h_1 = 8{\cdot}29$ and $h_2 = 15$ feet, would, according to that formula, require a depth of water-cushion of only 108 feet for brickwork, or 72 for stone, a depth nearly a half less than the actual depth, 130 feet.

In a smaller natural case, in hills in Berar, coming under the observation of the author, for values $h_1 = 26$ and $h_2 = 1$, the depth, according to the above formulæ, would be for a brickwork bottom 7·65 feet, and for stone 5·6 feet; whereas, in the soundest of basalt, the actual depth was as much as 8 feet, or more than a quarter more than that calculated.

It would, therefore, appear that the above formula, apart from its varied coefficients for brickwork and stone, is generally defective, and that, until a very much wider range of experiments and observations is made, it would be more advisable to approximate to such depths as are obtained under natural conditions, than to follow any formula for determining the depth of a basin serving as a water-cushion.

In practice it would rarely be necessary to construct a water-cushion of very great depth, the fall, if over a weir, being generally easily broken into three or four portions, and it being advantageous to do so, as the catch channels are convenient for affording a supply at various levels; probably, therefore, the above-mentioned case of 43·5 feet of artificial fall may be considered as the extreme for which a water-cushion would be required. In the future, too, the waste of such a large amount of useful motive power will be deemed a barbarism, an additional reason that there is not much probability of the above case being exceeded.

9.—THE USUAL THICKNESS OF WATER-PIPES.

The thickness of a water-pipe is a matter depending on practical considerations, being comparatively little affected by the theoretical determination of what it should be in order to resist the pressure brought on it; and is, like a very large number of the so-called calculations of the engineer, made almost entirely dependent on prescribed custom. The following notes on the formulæ in vogue are, hence, not given so much with the object of elucidating the principles as that the formulæ themselves, valueless as they seem, should be available for reference.

The largest scale on which a water-pipe to resist extreme internal pressure is made is that of the cylinders of hydraulic presses: in these the extreme working pressure is limited to 4 tons per square inch, the extreme permanent strain allowed in actual working being only one half of that; and the thickness of the cylinder or pipe is determined by the formula of Barlow—

$$t = \frac{r.\ P}{C - P};$$

where t and r are the thickness and internal radius of the cylinder or pipe,

C is the cohesive strength of the material, and

P is the internal pressure, both being in tons:

the general principle asserted in this mode of calculation being that the strain on the material is greatest at the internal surface, and less beyond, the extension varying with the square of the distance from the centre.

An example of the application of this formula, to a 10-inch cast-iron water-pipe, is given in Box's "Hydraulics," the results of which are as follows:—

Assuming the cohesive strength of cast iron to be 7 tons per square inch breaking weight; the extension E, on the inside ring at the moment of rupture, for a length = 1,

$$E = \cdot 000\,165\ W + \cdot 000\,010\,3\ W^2 \times L = \cdot 001\,659\,7;$$

and the extension at any distance from the centre is in the ratio of the square of that distance to that of the inside ring.

The strain, at any distance from the centre, is then obtained from the extension by the formula—

$$W = \sqrt{\left(\frac{E}{\cdot000\,0103 \times L} + 64{\cdot}16\right)} - 8{\cdot}01$$

and the mean strain on each theoretical concentric ring of metal is the average between that at its external and its internal circumference; the bursting pressure has then the same ratio to the mean strain as the thickness of the pipe has to its radius; and tabulating these for a 10-inch cast-iron pipe, they are:—

Thickness of Metal.	Strain on the Metal.			Bursting Pressure.
	Max.	Min.	Mean.	
1″	7.0	5·26	6·130	1·226
2	7·0	4·09	5·402	2·161
3	7·0	3·26	4·827	2·896
4	7·0	2·65	4·359	3·485
5	7·0	2·20	3·972	3·972
6	7·0	1·85	3·647	4·337
7	7·0	1·60	3·373	4·722
8	7·0	1·37	3·137	5·019
9	7·0	1·19	2·931	5·275
10	7·0	1·05	2·749	5·499

The practical empirical rule, however, that is given by Box for the thicknesses of water-pipes is—

$$t = \left(\sqrt{\frac{d}{10}} + 0{\cdot}15\right) + \left(\frac{H\,d}{25\,000}\right);$$

where H is the head of pressure, and d is the diameter of the pipe, and it is according to this, that his table given in the Appendix of Miscellaneous Tables is calculated.

The theoretical mode of arriving at the thickness of a water-pipe is, therefore, about the most unsatisfactory of processes; and it would probably be useless to enlarge on the topic. In actual practice, the dimensions of cast-iron water-pipes are about those given in Box's table; or have a thickness of one-fifth the square root of the diameter, and a little more to allow for defects in casting, and inexactitude of bore. The dimensions of the details of the sockets are also given in the second part of Box's table, and are very convenient for reference.

Flanged pipes being now so rarely used, excepting for temporary purposes, the details of their usual dimensions and weights, given by Box, are omitted in the table given.

While in the case of cast-iron pipes of all sorts, there has always been a tendency to theorise, and to base a thickness on the laws of pressure, and extension of material; in stoneware pipes, this has been almost entirely disregarded, and a thickness is generally given them that is established entirely on practice or usual custom, and often varies according to the caprice of the potter or manufacturer. This is generally accounted for by saying that earthenware or stoneware is a very variable material as regards strength, while cast iron is homogeneous, and is very much alike in substance: a little reflection, however, will show that this is hardly a sufficient reason. Carefully made stoneware, after a very careful selection, may be, and often is, exceedingly equable, while the variety of qualities of cast iron, —more especially since its high price has brought such a large amount of very inferior material into use,—is now very marked; some cast iron being known occasionally to fall to pieces from its own weight. In spite of this, the manufacturers of stoneware pipes still consider them as unsuited to the discharge of water under pressure, or for drainage in cases where the outlet is liable to be stopped: and although they can make pipes that will easily bear a head of 40 feet, yet do not recommend them, alleging that the joints cannot be made to stand any pressure at all. There is, however, no reason to doubt that under skilled superintendence and management, stoneware and fire-clay pipes, as well as their joints, may be well enough made to serve most efficiently for the distribution and drainage of water under low heads, and that a considerable saving of expense may be effected by dispensing with iron in such cases.

10.—INDIAN HYDRAULIC CONTRIVANCES.

In India a large variety of mechanical contrivances of a very simple nature are commonly used for raising water from rivers or wells or out of foundations of bridges, that are generally unknown

to the English engineer. His natural tendency would be to use the appliances best known to him, such as a windlass and bucket, a common pump, a lift and force-pump, or a winding-up chain carrying iron vessels; of these the last only is very well known in India in a more simple form, as a chain of pots or leather bags. Pumps are purely European in origin, even a windlass is a comparative rarity; and since such things are not always available it often becomes necessary for him to adopt the native means of raising water and to learn what duty may be expected from them. To aid him, or rather to save him needless trouble in measuring and calculating the duty, the table given in the Appendix to the Working Tables, based upon data originally furnished by M. Lamairesse, of Pondicherry, for Southern India, and in the Roorkee professional papers for Northern India, and in conjunction with others by the author, but modified and put in a form intelligible to the English civil engineer, may be found useful. It merely becomes necessary to give the meaning of a few of the Indian names of the contrivances, and state the mode in which they are used.

Baling is one of the most primitive methods of raising water, but the English mode of filling and emptying a vessel or a bucket is not in vogue among the natives of India. A large flat dish of wood bark rendered water-tight, or leather stiffened by a frame, has two long cords attached to it at opposite sides, the other two ends of the cords being held by two men, who generally prefer sitting down to their work, and together allow the dish to dip in the water, nearly fill itself, and then raise it, send it forward with a swing and let it empty itself above; this can be done with a rapid and continuous swinging motion that is sometimes quite surprising. This method is of course only applicable under certain conditions, such as clearing foundations of water, and such cases as allow of sufficient room for the swinging; the lift is seldom more than 5 ft. though sometimes 7 ft.; but a series of such lifts can be easily adopted.

The beam and bucket, or balance-pole, in its various modifications, is also a favourite contrivance for raising water from wells by hand labour; the lever, at one end of which is hung the water vessel, generally a large earthenware pot, is counter-weighted at

the other end so as just to allow the force of one man to raise the vessel when full. The lever is often a beam naturally very thick at one end, and requiring only to be carefully hung or supported at the most convenient point for a fulcrum. In Southern India this principle reaches its fullest development in the picotah; where a very large long tree, or a very large pair of trees bound together, becomes the balance-pole, to work which a man walks and runs backwards and forwards along the heavier arm of the lever, stepping off, when necessary, on to a raised stage; for this work special men, thoroughly accustomed to it, are absolutely necessary; one managing the vessel, the other the balancing. The size of these picotahs is sometimes extremely large, and the lift consequently very high.

The dal or jantu is a contrivance for raising water from 3 ft. to 6 ft. high by means of a wooden gutter moving on a pivot, being a lever, or a double lever of the second order. There are several forms of this contrivance; in the simplest, one end of the single gutter is raised by a man with a cord or lever and cord, until the water runs out of the other end of the gutter into a trench; in the double gutter there is a wooden partition in the gutter immediately above the pivot, and the water runs out through holes on each side of it in the bottom of the gutter into the trench; sometimes these are worked by cords, and sometimes by means of the weight of a man and a counterpoise at the end of a long lever attached.

The mot is an arrangement worked by oxen; it generally consists in a water vessel made of a complete ox-hide bound on to a wooden ring for an opening, raised and lowered by a cord running over a pulley, and fixed immediately above or projecting over the well; the bullocks going down an inclined plane made for the purpose, when dragging up the water vessel or mot, which has to be dragged to one side on arrival above the mouth of the well and emptied by a man. In Southern India there is an improvement on this which dispenses with the man for emptying; the lower end of the mot tapers out to a considerable length, and has a smaller cord attached to it, which by means of a suitably adjusted catch causes the mot to empty itself on arriving at the proper height.

The contrivance generally called by Anglo-Indians a Persian wheel, but more properly a chain of pots, is almost identical with

that used in Egypt, Nubia, Syria, Abyssinia, known there as the sakia; its advantage is that it will raise water from any depth by means of sufficient animal power. In India it is generally very much of the following description. Two parallel endless ropes, united to each other by rungs of wood or of rope, pass over a vertical wheel and hang down to below the water surface in the well; earthen or leathern vessels are attached to the rungs, which discharge themselves into a trough through the vertical wheel, which is a double frame-work. Motion is communicated to the axle of this vertical wheel from a vertical shaft of wood that is turned by a pair of bullocks, by means of two wooden wheels working into each other. The upper end of the vertical shaft is kept in position by a very heavy beam or tree which rests also on two supports, generally mud walls, beyond the sweep of the circle in which the oxen walk. The principle of this rather rude but effective contrivance was doubtless the basis of the double iron chains of pots, with brass buckets holding about a gallon each, that were used by the Romans, and hence also the remote ancestor of our modern chains of pots having chains of jointed iron bars, skeleton six-spoked or hexagonal wheels, and buckets or iron casks of the most improved form; or again, somewhat like those used and worked by steam power on the Metropolitan District Railway to clear the line of water.

The true Persian wheel, with which the chain of pots is sometimes confounded, is a wheel with a hollow tyre, and is an inferior contrivance, suitable only to small lifts.

Referring to the table given, the details of which have been reduced and modified in order to show as much as possible what comparison may be drawn in favour of each machine, it will be noticed that the full amount of work done and power exerted is, in the first place, given for all cases, under a theoretical condition that never occurs in practice. In each and all of these machines, a certain amount of work is wasted by leakage, spilling, faulty construction, or inexactness of form, delay for small repairs, and many other such causes. To obtain anything near the truth, therefore, a coefficient of reduction that is purely empirical must be applied. Some of these coefficients are given in the Roorkee professional papers, others are obtained from other sources; they may for our

purposes in dealing with such rough machines be applied equally to the work done and the effective power exerted; but as the latter is the principal object under consideration, the amounts under that head only are reduced. The final quantities, therefore, are more practically useful.

The results may not at first sight appear to admit of much comparison being made; certain things are, however, plainly indicated by them, the most marked one being that all such rough hydraulic contrivances used in small lifts involve a great waste of power as well as of water, much intermediate time being lost between the lifts, and that the machine itself, when on a large scale, being more properly made and more carefully worked is far more effective. This is shown most on comparing the effective results of the North Indian beam and bucket (1·2) with the Southern Indian picotahs (1, 2, 3); in the mots, on the other hand, the advantage is on the side of the North Indian, probably from his using an additional man, although it is probably obtained at too great an expense. The chain of pots more exclusively used in Northern India appears to be, under theoretical conditions, the most effective of all these contrivances. The data given are, from the very variable nature of such things, too rough to allow of any comparison being drawn between such contrivances and the more civilized arrangements; but they may, however, be of use to those unacquainted with Indian contrivances when first called on to deal with them.

HYDRAULIC WORKING TABLES.

ERRATA.

p. lxxxii *for* $\frac{D}{W} = \cdot 1$ *read* $\frac{D}{W} = 1$.

p. lxxxiii *for* ·26 *read* ·62.

for $\left(1 \times \frac{h}{H}\right)^{\frac{3}{2}}$ *read* $\left(1 + \frac{h}{H}\right)^{\frac{3}{2}}$.

p. lxxxv *twice* for ounces *read* drams.

for $P \times 2{\cdot}31$ *read* $P \times 0{\cdot}016$.

b

HYDRAULIC WORKING TABLES.

I. Gravity.
II. Catchment.
III. Storage and Supply.
IV. Flood Discharge.
V. Velocities.
VI. Slopes and Gradients.
VII. Rivers and Canals.
VIII. Pipes and Sewers.
IX. Sluices and Weirs.
X. Bends and Obstructions.
XI. Equivalents.
XII. Coefficients and Memoranda.

APPENDIX.

Miscellaneous Tables.

b

TABLE I.

Values of the force of gravity in feet for different latitudes and elevations above mean sea level.

ELEVATION.	LATITUDE.								
	20°	25°	30°	35°	40°	45°	50°	55°	60°
0	32·0994	32·1108	32·1238	32·1383	32·1536	32·1695	32·1854	32·2008	32·2152
100	32·0991	32·1105	32·1235	32·1380	32·1533	32·1692	32·1851	32·2005	32·2149
200	32·0988	32·1102	32·1232	32·1377	32·1530	32·1689	32·1848	32·2002	32·2146
300	32·0985	32·1099	32·1229	32·1374	32·1528	32·1686	32·1845	32·1998	32·2143
400	32·0982	32·1096	32·1226	32·1371	32·1524	32·1683	32·1842	32·1995	32·2140
500	32·0979	32·1093	32·1223	32·1368	32·1521	32·1680	32·1839	32·1992	32·2137
600	32·0976	32·1090	32·1220	32·1364	32·1518	32·1677	32·1835	32·1989	32·2134
700	32·0973	32·1087	32·1217	32·1361	32·1515	32·1674	32·1832	32·1986	32 2131
800	32·0970	32·1084	32·1214	32·1358	32·1512	32·1671	32·1829	32·1983	32·2128
900	32·0967	32·1081	32·1211	32·1355	32·1509	32·1668	32·1826	32·1980	32·2125
1000	32·0963	32·1077	32·1208	32·1352	32·1506	32·1665	32·1823	32·1977	32·2121
2000	32·0933	32·1047	32·1177	32·1322	32·1473	32·1633	32·1793	32·1947	32·2091
3000	32·0902	32·1017	32·1146	32·1291	32·1442	32·1603	32·1762	32·1916	32·2060
4000	32·0871	32·0986	32·1115	32·1260	32·1411	32·1572	32·1731	32·1885	32·2029
5000	32·0840	32·0955	32·1084	32·1229	32·1382	32·1541	32·1700	32·1854	32·1998

Being an application of the formulæ $g = 32{\cdot}1695\,(1 - {\cdot}00284 \cos 2l)\left(1 - \frac{2e}{r}\right)$

$r = 20887540\,(1 + {\cdot}00164 \cos 2l)$

TABLE II.—Part 1.

Total quantities of water equivalent to a given rainfall.

Rainfall in feet.	Cubic feet per acre.	Cubic feet per square mile.	Rainfall in feet.	Cubic feet per acre.	Cubic feet per square mile.
1·	43 560	27 870 400	(12″) 1·	43 560	27 878 400
·9	39 204	25 090 560	(11″) ·917	39 900	25 555 200
·8	34 848	22 302 720	(10″) ·833	36 300	23 232 000
·7	30 492	19 514 880	(9″) ·750	32 670	20 908 800
·6	26 136	16 727 040	(8″) ·666	29 040	18 505 600
·5	21 780	13 939 200	(7″) ·583	25 410	16 262 400
·4	17 424	11 151 360	(6″) ·5	21 780	13 939 200
·3	13 068	8 363 520	(5″) ·417	18 150	11 616 000
·2	8 712	5 575 680	(4″) ·333	14 520	9 252 800
·1	4 356	2 787 840	(3″) ·250	10 890	6 969 600
			(2″) ·166	7 260	4 646 400
			(1″) ·083	3 630	2 323 200
·09	3 920	2 509 056			
·08	3 485	2 230 272			
·07	3 049	1 951 488			
·06	2 614	1 672 704			
·05	2 178	1 393 920			
·04	1 742	1 115 136			
·03	1 307	836 352			
·02	871	557 568			
·01	436	278 784			

For decimals of an inch of rainfall remove the point in the corresponding quantities.

N.B.—One square mile = 640 acres = 27,878,400 square feet.

TABLE II.—Part 2.

Discharges in cubic feet per second throughout the year, equivalent to a given annual rainfall over one square mile of catchment area.

Annual rainfall in feet.	Discharges in cubic feet per second.	Annual rainfall in feet.	Discharges in cubic feet per second.	Annual rainfall in feet.	Discharges in cubic feet per second.
·1	·0883	2·1	1·8550	4·1	3·6217
·2	·1766	2·2	1·9433	4·2	3·7100
·3	·2650	2·3	2·0317	4·3	3·7983
·4	·3533	2·4	2·1200	4·4	3·8866
·5	·4417	2·5	2 2083	4·5	3·9750
·6	·5300	2·6	2·2966	4·6	4·0633
·7	·6183	2·7	2·3850	4·7	4·1517
·8	·7066	2·8	2·4733	4·8	4 2400
·9	·7950	2·9	2·5617	4·9	4·3283
1·0	·8833	3·0	2·6500	5·0	4·4166
1·1	·9717	3·1	2·7383	5 5	4·8583
1·2	1·0600	3·2	2·8266	6·	5·3000
1·3	1·1483	3·3	2·9150	6·5	5·7417
1·4	1·2366	3·4	3·0033	7·	6·1833
1·5	1·3250	3·5	3·0917	7·5	6·6250
1·6	1·4133	3·6	3·1800	8·	7·0666
1·7	1·5017	3·7	3·2683	8·5	7·5083
1·8	1·5900	3·8	3·3566	9·	7·9500
1·9	1·6783	3·9	3·4450	9·5	8·3917
2·0	1·7666	4·0	3·5333	10·	8·8333

TABLE II.—Part 3.

Discharges in cubic feet per second, equivalent to a given daily rainfall (24 hours) over catchment areas.

Area in sq. miles.	For a daily rainfall in feet and decimals of									
	·1	·09	·08	·07	·06	·05	·04	·03	·02	·01
	Cubic feet per second.									
1	32·26	29·03	25·81	22·58	19·34	16·13	12·90	9·67	6·45	3·23
2	64·52	58·07	51·62	45·16	38·72	32·26	25·81	19·36	12·90	6·45
3	92·80	83·52	74·24	64·96	55·68	48·40	37·12	27·84	18·56	9·28
4	129·0	116·1	103·2	90·30	76·40	64·50	51·60	38·70	25·80	12·90
5	161·3	145·2	129·0	112·9	96·80	80·64	64·50	48·40	32·25	16·13
6	193·5	174·2	154·8	135·4	116·1	96·78	77·40	58·06	38·70	19·35
7	225·8	203·2	180·6	158·0	135·5	112·9	90·30	67·73	45·15	22·58
8	258·0	232·2	206·4	180·6	154·8	129·0	103·2	77·40	51·60	25·80
9	290·4	261·4	232·3	203·3	174·3	145·2	116·2	87·13	58·10	29·04
10	322·6	290·3	258·3	225·8	193·5	161·3	129·2	96·77	64·60	32·26

Area in square miles.	For a daily rainfall in feet and decimals of									
	·0833	·075	·0666	·0583	·05	·0417	·033	·025	·016	·0083
	or its equivalent in inches and decimals of									
	1·0	·9	·8	·7	·6	·5	·4	·3	·2	·1
	Cubic feet per second.									
1	26·89	24·20	21·51	18·82	16·13	13·44	10·75	8·07	5·38	2·69
2	53·78	48·40	43·00	37·64	32·26	26·89	21·50	16·13	10·75	5·38
3	80·67	54·60	64·53	56·47	48·40	40·33	32·26	24·20	16·13	8·07
4	107·5	96·75	86·00	75·25	64·50	53·78	43·00	32·25	21·50	10·75
5	134·4	120·9	107·5	94·08	80·64	67·22	53·75	40·32	26·87	13·14
6	161·3	145·1	135·0	112·9	96·78	80·67	67·55	48·39	33·77	16·13
7	188·2	169·3	150·5	131·7	112·9	94·11	75·25	56·45	37·62	18·82
8	215·1	193·6	172·1	150·5	129·0	107·5	86·05	64·50	43·02	21·51
9	242·0	217·8	193·6	169·4	145·2	121·0	96·80	72·60	48·40	24·20
10	268·9	242·0	215·1	187·4	161·3	134·4	107·5	80·65	53·75	26·89

TABLE III.—Part 1.

Guide for capacity of reservoirs and supply from gathering grounds.

Supply required, during 240 days or eight months.	Contents of reservoir to hold that supply.	Surface of that reservoir if 3 feet deep on the average.	Catchment area necessary to fill that reservoir in four months, having one foot available rainfall in that time.
Cub. ft. per second.	Cubic feet.	Square feet.	Square miles.
1	20 736 000	6 912 000	·7438
2	41 472 000	13 824 000	1·4876
3	62 208 000	20 736 000	2·2314
4	82 944 000	27 648 000	2·9752
5	103 680 000	34 560 000	3·7190
6	124 416 000	41 472 000	4·4628
7	145 152 000	48 384 000	5·2066
8	165 888 000	55 296 000	5·9504
9	186 624 000	62 208 000	6·6942
10	207 360 000	69 120 000	7·4380
1·3444	27 878 400	9 292 800	1
2·6888	55 756 800	18 585 600	2
4·0333	83 635 200	27 878 400	3
5·3777	111 513 600	37 171 200	4
6·7222	139 392 000	46 464 000	5
8·0666	167 270 400	55 756 800	6
9·4100	195 148 800	65 049 600	7
10·7555	223 027 200	74 342 000	8
12·0999	250 905 600	83 635 200	9
13·4444	278 784 000	92 928 000	10

N.B.—The next page will contain two examples for this table.

EXAMPLE I.

A discharge of 18 234 cubic feet per second is wanted during eight months of the year from a reservoir which is to be supplied by a catchment area yielding an available rainfall of 1·32 feet during the remaining four months; required the contents of the reservoir, and the size of the catchment area.

Obtain from the Table the quantities due to 1 foot of rainfall,

Supply, cubic feet per second.	Contents of reservoir, cubic feet.	Catchment area, square miles.
10	207 360 000	7·4380
8	165 888 000	5·9504
·2	4 147 200	·1487
·03	622 080	·0223
·004	82 944	·0029
18·234	378 100 224	3·5623

Catchment area for 1·32 feet of fall $= \frac{13 \cdot 5623}{1 \cdot 32} = 10 \cdot 274$ sq. miles.

EXAMPLE II.

A catchment area of 21·963 square miles, having an available rainfall of 1·32 feet in four months of rainy season, supplies a reservoir which is to hold water for eight months' supply; what should be the full contents of the reservoir, and the supply in cubic feet per second during the eight months?

The proportionate catchment area for an available rainfall of one foot will $= 21 \cdot 963 \times 1 \cdot 32 = 29 \cdot 001$ square miles.

Area	Contents of reservoir, cubic feet.	Supply, cub. ft. per second.
20	557 568 000	26·888
9	250 905 600	12·0999
·001	27 878	·0013
29·001	808 501 478	38·9892

TABLE III.—Part 2.

Guide for acreage under irrigation, and for population under water-supply.

Cub. feet per second.	At 50 acres per cub. foot per second.	At 75 acres per cub. foot per second.	At 100 acres per cub. ft. per second.	At 150 acres per cub. ft. per second.	At 200 acres per cub. ft. per second.	At 250 acres per cub. ft. per second.	At 300 acres per cub. ft. per second.
	Number of acres watered.						
1	50	75	100	150	200	250	300
2	100	150	200	300	400	500	600
3	150	225	300	450	600	750	900
4	200	300	400	600	800	1000	1200
5	250	375	500	750	1000	1250	1500
6	300	450	600	900	1200	1500	1800
7	350	525	700	1050	1400	1750	2100
8	400	600	800	1200	1600	2000	2400
9	450	675	900	1350	1800	2250	2700
10	500	750	1000	1500	2000	2500	3000

Cub. feet per second.	At 5 gallons per head daily.	At 7½ gallons per head daily.	At 10 gallons per head daily.	At 15 gallons per head daily.	At 20 gallons per head daily.	At 25 gallons per head daily.	At 30 gallons per head daily.
	Population supplied.						
1	107732	71820	53866	35910	26933	21546	17955
2	215464	143640	107732	71820	53866	43093	35910
3	323196	215460	161598	107730	80799	64639	53865
4	430928	287280	215464	143640	107732	86186	71820
5	538660	359100	269330	179550	134665	107932	89775
6	646392	430920	323196	215460	161598	129278	107730
7	754124	474740	377062	237370	188531	150825	118685
8	861856	574560	430928	287280	215464	172371	143640
9	969588	646380	484794	323190	242397	193917	161595
10	1077320	718200	538660	359100	269330	215464	179550

N.B.—The next page will contain explanatory examples.

Example I.

A combined irrigation and water-work scheme yields 18·234 cubic feet per second; what amount of land and of population could it supply, at the rates of 150 acres per cubic foot per second, and 7½ gallons per head per diem, if one-fourth alone is to be used for the water-works?

The supply available for irrigation will be = 18·234 − 4·558 = 13·676 cubic feet per second; and from Table III., Part 2, we obtain the required results, thus—

Cubic feet per second.	Population.	Cubic feet per second.	Acres.
4.	287 280	10·	1500
·5	35 910	3·	450
·05	0 591	·6	90
·008	574	·07	10·5
		·006	·9
4·558	327 355	13·676	2051·

Example II.

A town has a population of 40,000, requiring water supply at 15 gallons per head daily, and has suburbs to the extent of 1,400 acres requiring irrigation at 150 acres per cubic foot per second of supply:—what catchment area will be necessary to provide this, if the annual rainfall is 60 inches?

According to Part 2, Table III., the supply necessary will be

For population.		For irrigation.		Total cubic feet per second.
35 910	1	1 350	9·	
3 591	·1	50	·04	
489	·02			
40 000	1·12	1 400	9·04	10·16

Now, assuming that out of 60 inches annual rainfall, 30 can be utilized after deducting for all losses:—we find that according to Part 2, Table II., this is equivalent to a supply of 2·2083 cubic feet per second from one square mile, hence the minimum catchment area necessary will $= \dfrac{10{\cdot}16}{2{\cdot}208} = 4{\cdot}6$ square miles.

TABLE IV.—Part 1.

Table of flood discharges in cubic feet per second, due to catchment areas in square miles, and corresponding to a coefficient $n=1$ in the formula

$$Q = n \times 100\,(N)^{\frac{3}{4}}.$$

For local values of coefficients, see Part 2, Table XII.

Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.
·01	3	11	604	41	1620	71	2446
·02	5	12	645	42	1650	72	2472
·03	7	13	685	43	1679	73	2498
·04	9	14	724	44	1708	74	2523
·05	11	15	762	45	1737	75	2549
·06	12	16	800	46	1766	76	2574
·07	14	17	837	47	1795	77	2599
·08	15	18	874	48	1824	78	2625
·09	16	19	910	49	1852	79	2650
		20	946	50	1880	80	2675
·1	18	21	981	51	1908	81	2700
·2	30	22	1016	52	1936	82	2725
·3	41	23	1050	53	1964	83	2750
·4	50	24	1084	54	1992	84	2775
·5	59	25	1118	55	2020	85	2799
·6	68	26	1151	56	2047	86	2824
·7	76	27	1184	57	2074	87	2849
·8	85	28	1217	58	2802	88	2873
·9	92	29	1250	59	2129	89	2898
		30	1282	60	2155	90	2922
1·	100	31	1314	61	2183	91	2946
2·	168	32	1345	62	2210	92	2971
3·	238	33	1377	63	2236	93	2995
4·	283	34	1408	64	2263	94	3019
5·	334	35	1439	65	2289	95	3043
6·	383	36	1470	66	2316	96	3067
7·	430	37	1500	67	2342	97	3091
8·	476	38	1531	68	2368	98	3115
9·	520	39	1561	69	2394	99	3139
10·	562	40	1590	70	2420	100	3162

TABLE IV.—PART 1—*continued.*

Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.	Catchment area.	Flood discharge.
110	3397	410	9112	710	13 754	1250	21 022
120	3625	420	9278	720	13 900	1500	24 103
130	3850	430	9443	730	14 044	1750	27 057
140	4070	440	9607	740	14 188	2000	29 907
150	4286	450	9770	750	14 332	2500	35 355
160	4499	460	9933	760	14 475	3000	40 536
170	4708	470	10 094	770	14 617	3500	45 504
180	4914	480	10 255	780	14 760	4000	50 297
190	5117	490	10 415	790	14 901	4500	54 943
200	5318	500	10 574	800	15 042	5000	59 460
210	5517	510	10 732	810	15 183	5500	63 867
220	5712	520	10 890	820	15 324	6000	68 178
230	5906	530	11 046	830	15 463	6500	72 391
240	6098	540	11 202	840	15 603	7000	76 529
250	6287	550	11 357	850	15 742	7500	80 593
260	6475	560	11 512	860	15 881	8000	84 590
270	6661	570	11 666	870	16 019	8500	88 525
280	6845	580	11 819	880	16 157	9000	92 402
290	7027	590	11 791	890	16 295	9500	96 448
300	7208	600	12 123	900	16 432	10 000	100 000
310	7388	610	12 204	910	16 568		
320	7566	620	12 425	920	16 705	20 000	168 179
330	7743	630	12 575	930	16 841	30 000	238 285
340	7918	640	12 724	940	16 976	40 000	282 355
350	8092	650	12 873	950	17 112	50 000	334 370
360	8265	660	13 021	960	17 246	60 000	383 366
370	8436	670	13 169	970	17 381	70 000	430 352
380	8607	680	13 316	980	17 511	80 000	475 683
390	8776	690	13 463	990	17 649	90 000	519 615
400	8944	700	13 609	1000	17 783	100 000	562 341

TABLE IV.—Part 2.

Flood discharges from catchment areas with a coefficient $n = 8{\cdot}25$ and corresponding waterway for bridge openings.
(By Colonel Dickens.)

Catchment area.	Flood discharge, co-eff 8·25	Assumed velocity.	Flood waterway.	No. of sq. openings.	Span.	Height of pier.
Square miles.	Cub. feet per sec.	Feet.	Square feet.	No.	Feet.	Feet.
·0016	6·5	5	1·5	1	1½	1
·0031	11·	5	2·25	1	2	1⅛
·0047	15	5	3·	1	2	1½
·0078	22	5	4·5	1	3	1½
·0125	31	5	6·	1	3	2
·0250	52	5	10·5	1	4	2¾
·0625	103	6	18·	1	6	3
·1250	173	6	29·	1	7	4
·2500	292	6	49·	1	10	5
·5000	490	6	81·	1	12	7
1	825	7	137	2	12	6
2	1 388	7	200	3	12	6
3	1 881	7	270	3	14	7
5	2 760	7	400	3	16	8
7	3 550	7	507	3	18	9
10	4 640	7	663	3	20	11
20	7 804	8	975	5	20	10
30	10 577	8	1 322	5	24	11
50	15 605	9	1 734	5	30	11½
100	26 094	9	2 899	5	40	14½
200	43 884	10	4 388	7	40	15½
300	59 481	10	5 948	9	40	16½
500	87 255	10	8 725	9	50	19
1 000	146 737	10	14 673	15	50	19
2 000	246 780	11	22 434	15	60	24
3 000	334 487	11	30 408	20	60	25
5 000	490 636	12	40 886	20	75	27
10 000	825 000	12	68 750	30	75	30
20 000	1 387 746	13	106 749	40	75	35
30 000	1 870 962	13	143 920	45	80	40
50 000	2 695 690	14	190 256	50	90	42
100 000	4 639 274	15	309 285	60	100	50

TABLE V.

Comparative, usual, and safe bottom velocities.

	Feet per second.		Feet per second.
Slow rivers	·33	Sailing ships ...	17
Ordinary rivers ...	2·25	Sea steamers ...	20
Rapid rivers	10·25	River steamers ...	26
A man's walk ...	4·5	Railways, English ...	47
Horse trot	10·25	" French ...	40
Racing speed ...	88	" American	27
Winds	10·25	Sound at 30° ...	1090
Storms	52·75	Sound at 63° ...	1122
Hurricanes	117·25	Air into a vacuum. Bar. 30″	1344

	Feet per second.
Limits usual for canals	1 to 4
Limits for rivers and canals just navigable	3 to 4½
Limits for irrigating channels	1 to 3
Limits for sewers and brick conduits	1 to 4½.
Earthenware drainage pipes	4½
Maximum tidal current measured	15
Best velocities for pipes, so as to get a maximum discharge under pressure	25 to 35

Safe maximum bottom velocities.	Feet per second.
For soft clay	·25
For fine sand	·5
For coarse sand and small gravel	·7
For gravel as large as beans	1·
For gravel one inch in diameter	2·25
For pebbles one and a half inches in diameter	3·33
For heavy shingle	4·
For softer rocks, brick and earthenware	4·5
For hard rock	6 to 10

TABLE VI.—Part 1.

Ordinary limits of channel gradients.

Reciprocal of slope.	
1 in 500 000	Least canal slope to produce motion.
1 in 16 000 1 in 6 000	Limits of tidal navigation for large canals.
1 in 15 000 1 in 5 000	Fall of most deltaic or inundation canals.
1 in 6 000 1 in 2 000	Fall of most canals.
1 in 3 000 1 in 1 000	Fall of smaller canals, channels.
1 in 5 000 1 in 500	Fall of most rivers.
1 in 300 1 in 80	Fall of torrents.

Maximum gradients.

1 in 50	Ordinary railways.
1 in 30	Turnpike road.
1 in 20	Public road.
1 in 16	Private road.
1 in 8	Maximum for an ordinary carriage to ascend.
1 in 4	Maximum for beasts of burden.
1 in 1½	Maximum for hill walking.

Various slopes.

1 to 1 to ½ to 1	Chalk; dry clay.
1 to 1	Compact earth rubble, dry set.
1¼ to 1	Gravel, shingle, dry sand.
1½ to 1	Average mixed earth, dry.
1¾ to 1	Vegetable earth, dry.
2 to 1	Sand dry.
2 to 1	Minimum for slated and tiled roofs.
2½ to 1	Maximum for back slopes of rammed earthen dams.
3½ to 1	Maximum for breast slopes of rammed earthen dams.
4 to 1 to 3 to 1	Wet clay, peat.

N.B.—Wetted soil requires a less slope than dry soil generally.

TABLE VI.—Part 2.

Reduction of gradients.

Slope (S)	Fall of one in	Fall in feet per mile.	Slope (S)	Fall of one in	Fall in feet per mile.
·000 0100	100 000	·0528	·000 55	1818	2·904
·000 0133	75 000	·0704	·000 6	1666	3·168
·000 0150	66 666	·0792	·000 65	1538	3·332
·000 0200	50 000	·1056	·000 66	1500	3.52
·000 0250	40 000	·1320	·000 7	1429	3·696
·000 0300	33 333	·1584	·000 75	1333	3·960
·000 0333	30 000	·1760	·000 8	1250	4·224
·000 0350	28 571	·1848	·000 85	1176	4·488
·000 0400	25 000	·2112	·000 9	1111	4·752
·000 0450	22 222	·2376	·000 95	1053	5·016
·000 0473	21 120	·2500			
·000 0500	20 000	·2640	·001	1000	5·28
·000 0600	16 666	·3168	·00110	909	5·808
·000 0700	14 286	·3696	·00111	900	5·864
·000 0800	12 500	·4224	·00125	800	6·6
·000 0900	11 111	·4752	·00143	700	7·54
·000 0947	10 560	·5	·0015	666	7·92
·000 1000	10 000	·528	·00166	600	8·8
·000 1111	9000	·5866	·00175	571	9·24
·000 1250	8000	·6600	·002	500	10·56
·000 1420	7004	·7500			
·000 1428	7000	·7543			
·000 1500	6666	·7920	·00225	444	11·88
·000 1666	6000	·8800	·0025	400	13·20
·000 1750	5714	·9240	·00275	364	14·52
·000 1894	5280	1·	·003	333	15·84
·000 2000	5000	1·056	·00325	308	16·66
			·00333	300	17·60
·000 25	4000	1·320	·0035	286	18·48
·000 3	3333	1·584	·00375	266	19·80
·000 333	3000	1·760	·004	250	21·12
·000 35	2857	1·848	·00425	235	22·44
·000 4	2500	2·112	·0045	222	23·76
·000 45	2222	2·376	·00475	210	25·08
·000 5	2000	2·640	·005	200	26 40

TABLE VI.—Part 2—*continued.*

Reduction of gradients.

Slope S.	Fall of one in	Fall in feet per mile.	Slope S.	Fall of one in	Fall in feet per mile.
·005	200	26·40	·015385	65·	81·23
·005263	190	27·78	·0155	64·5	81·84
·0055	181·8	29·04	·016	62·5	84·48
·005555	180	29·33	·0165	60·6	87·12
·005882	170	31·05	·016667	60·	88·
·006	166·66	31·68	·017	58·8	89·76
·006250	160	33·	·0175	57·1	92·40
·0065	153·8	33·32	·018	55·6	95·04
·006667	150	35·20	·018182	55·	96·
·007	142·86	36·96	·0185	54·1	97·68
·007143	140	37·71	·019	52·6	100·32
·0075	133·3	39·60	·0195	51·3	102·96
·007692	130	40·60	·02	50·	105·6
·008	125	42·25			
·008333	120	44·	·021	47.6	110·88
·0085	117·6	44·88	·022	45·4	116·16
·009	111·1	47·52	·023	43·5	121·44
·009091	110	48·	·024	41·7	126·72
·0095	105·3	50·16	·025	40·	132·
·01	100	52·80	·026	38·5	137·28
			·027	37·0	142·56
·0105	95·2	55·44	·028	35·7	147·84
·010526	95	55·57	·029	34·5	153·12
·011	90·9	58·08	·03	33·3	158·4
·011111	90	58·66			
·0115	86·9	60·72	·031	32·3	163·68
·011765	85	62·11	·032	31·3	168·96
·012	83·3	63·36	·033	30·3	174·24
·0125	80	66·	·034	29·4	179·52
·013	76·9	68·64	·035	28·5	184·8
·01333	75	70·40	·036	27·8	190·08
·0135	74·1	71·28	·037	27·0	195·36
·014	71·4	73·92	·038	26·3	200·64
·014286	70	75·42	·039	25·6	205·92
·015	66·7	79·20	·04	25·	211·2

TABLE VI.—PART 3.

Reduction table for angular slopes.

Angular Slope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.	Angular Slope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.
1°	57	·015	5° 30′	...	·460
1° 15′	46	...	5° 42′	10	...
1° 30′	39	·034	5° 45′	...	·503
1° 45′	33	...	6°	9·5	·548
2°	28	·061	6° 15′	...	·594
2° 15′	25	...	6° 21′	9	...
2° 30′	23	·095	6° 30′	...	·643
2° 45′	21	...	6° 43′	8·5	...
3°	19	·137	6° 45′	...	·693
3° 15′	18	·161	7°	...	·745
3° 28′	17	...	7° 7′	8	...
3° 30′	...	·187	7° 15′	...	·800
3° 35′	16	...	7° 30′	...	·856
3° 45′	...	·214	7° 36′	7·5	...
3° 49′	15	...	7° 45′	...	·913
4°	...	·244	8°	...	·973
4° 6′	14	...	8° 8′	7	...
4° 15′	...	·275	8° 15′	...	1·035
4° 24′	13	...	8° 30′	...	1·098
4° 30′	...	·308	8° 45′	6·5	1·164
4° 45′	12	·343	9°	...	1·231
5°	11·5	·381	9° 15′	...	1·300
5° 12′	11	...	9° 27′	6	...
5° 15′	...	·420	9° 30′	...	1·371
5° 27′	10·5	...	9° 45′	...	1·444

TABLE VI.—PART 3—*continued.*

Reduction table for angular slopes.

Angular Slope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.	Angular Slope.	Ratio to one perpendicular.	Reduction in feet and decimals for 100 feet.
9° 52′	5·75	...	17° 6′	3·25	...
10°	...	1·519	17° 30′	...	4·628
10° 18′	5·5	...	18°	...	4·894
10° 30′	...	1·675	18° 26′	3	...
10° 47′	5·25	...	18° 30′	...	5·168
11°	...	1·837	19°	...	5·448
11° 19′	5	...	19° 30′	...	5·736
11° 30′	...	2·008	19° 59′	2·75	...
11° 53′	4·75	...	20°	...	6·081
12°	...	2·185	21° 48′	2·5	...
12° 30′	...	2·370	23° 58′	2·25	...
12° 32′	4·5	...	25°	...	9·369
13°	...	2·553	26° 34′	2	...
13° 15′	4·25	...	29° 44′	1 75	...
13° 30′	...	2·763	30°	...	13·397
14°	4	2·970	33° 41′	1·5	...
14° 2′	...	...	35°	...	18·085
14° 30′	...	3·185	38° 39′	1·25	...
14° 55′	3·75	...	40°	...	23·396
15°	...	3·407	45°	1	...
15° 30′	...	3·637	50°	...	35·721
15° 56′	3·5	...	53° 8′	·75	...
16°	...	3·874	56° 20′	·66	...
16° 30′	...	4·118	60°	...	50·
17°	...	4·370	63° 26′	·5	...

TABLE VII.

Gives velocities of discharge for rivers, streams, and canals, for various hydraulic mean depths and slopes according to the formula.

$$V = c \times 100 \, (R. \, S)^{\frac{1}{2}} \text{ when } c = 1.$$

The tabular numbers extend the use of the table to any slope.

N.B.—For the use of co-efficients (c) see Part 3, Table XII.

TABLE VII.

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·000 33	·000 25	·0002	·000 166	·000 143	·000 125	·000 111	·0001
		Velocities of discharge in feet per second.									
·05	22·3607	·707	·5	·409	·353	·316	·289	·267	·25	·236	·224
·1	31·6228	1·	·707	·577	·5	·447	·408	·378	·353	·333	·316
·15	38·7298	1·225	·866	·707	·612	·547	·5	·463	·433	·408	·387
·20	44·7214	1·414	1·	·816	·707	·632	·577	·534	·5	·471	·447
·25	50·	1·581	1·118	·913	·790	·707	·645	·597	·559	·527	·5
·3	54·7723	1·732	1·225	·999	·866	·775	·706	·655	·612	·577	·548
·35	59·1608	1·871	1·323	1·081	·935	·837	·764	·707	·661	·624	·592
·4	63·2456	2·	1·414	1·154	1·	·894	·816	·756	·707	·666	·632
·45	67·0820	2·121	1·500	1·224	1·060	·949	·865	·802	·750	·707	·671
·5	70·7107	2·236	1·581	1·290	1·118	1·	·912	·845	·790	·745	·707
·55	74·1620	2·345	1·658	1·354	1·172	1·049	·957	·886	·829	·782	·742
·6	77·4597	2·449	1·732	1·431	1·224	1·095	1·	·926	·866	·816	·775
·65	80·6226	2·550	1·803	1·472	1·275	1·140	1·041	·964	·901	·850	·806
·7	83·6663	2·646	1·871	1·528	1·323	1·183	1·080	1·	·935	·882	·837
·75	86·6025	2·739	1·936	1·581	1·369	1·225	1·118	1·035	·968	·913	·866
·8	89·4427	2·828	2·	1·633	1·414	1·265	1·155	1·069	1·	·943	·894
·85	92·1955	2·915	2·062	1·683	1·457	1·304	1·190	1·101	1·031	·972	·922
·9	94·8683	3·	2·121	1·732	1.5	1·342	1·225	1·132	1·060	1·	·949
·95	97·4679	3·082	2·179	1·779	1·541	1·378	1·257	1·164	1·089	1·027	·975
1·00	100·	3·162	2·236	1·815	1·581	1·414	1·283	1·195	1·118	1·054	1·

TABLE VII.

Gives velocities of discharge for rivers, streams, and canals, for various hydraulic mean depths and slopes according to the formula.

$$V = c \times 100\ (R.\ S)^{\frac{1}{2}} \text{ when } c = 1.$$

The tabular numbers extend the use of the table to any slope.

N.B.—For the use of co-efficients (c) see Part 3, Table XII.

TABLE VII.

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·000 33	·000 25	·0002	·000 166	·000 143	·000 125	·000 111	·0001
		Velocities of discharge in feet per second.									
·05	22·3607	·707	·5	·409	·353	·316	·289	·267	·25	·236	·224
·1	31·6228	1·	·707	·577	·5	·447	·408	·378	·353	·333	·316
·15	38·7298	1·225	·866	·707	·612	·547	·5	·463	·433	·408	·387
·20	44·7214	1·414	1·	·816	·707	·632	·577	·534	·5	·471	·447
·25	50·	1·581	1·118	·913	·790	·707	·645	·597	·559	·527	·5
·3	54·7723	1·732	1·225	·999	·866	·775	·706	·655	·612	·577	·548
·35	59·1608	1·871	1·323	1·081	·935	·837	·764	·707	·661	·624	·592
·4	63·2456	2·	1·414	1·154	1·	·894	·816	·756	·707	·666	·632
·45	67·0820	2·121	1·500	1·224	1·060	·949	·865	·802	·750	·707	·671
·5	70·7107	2·236	1·581	1·290	1·118	1·	·912	·845	·790	·745	·707
·55	74·1620	2·345	1·658	1·354	1·172	1·049	·957	·886	·829	·782	·742
·6	77·4597	2·449	1·732	1·431	1·224	1·095	1·	·926	·866	·816	·775
·65	80·6226	2·550	1·803	1·472	1·275	1·140	1·041	·964	·901	·850	·806
·7	83·6663	2·646	1·871	1·528	1·323	1·183	1·080	1·	·935	·882	·837
·75	86·6025	2·739	1·936	1·581	1·369	1·225	1·118	1·035	·968	·913	·866
·8	89·4427	2·828	2·	1·633	1·414	1·265	1·155	1·069	1·	·943	·894
·85	92·1955	2·915	2·062	1·683	1·457	1·304	1·190	1·101	1·031	·972	·922
·9	94·8683	3·	2·121	1·732	1.5	1·342	1·225	1·132	1·060	1·	·949
·95	97·4679	3·082	2·179	1·779	1·541	1·378	1·257	1·164	1·089	1·027	·975
1·00	100·	3·162	2·236	1·815	1·581	1·414	1·283	1·195	1·118	1·054	1·

TABLE VII.—*continued*.

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
1·1	104·8809	3·317	2·345	1·915	1·658	1·483	1·354	1·254	1·172	1·106	1·049
1·2	109·5445	3·464	2·449	2·	1·732	1·549	1·414	1·310	1·224	1·155	1·095
1·3	114·0175	3·606	2·550	2·082	1·803	1·612	1·472	1·363	1·275	1·202	1·140
1·4	118·3216	3·742	2·646	2·160	1·871	1·673	1·527	1·414	1·323	1·247	1·183
1·5	122·4745	3·873	2·739	2·236	1·936	1·732	1·581	1·464	1·369	1·291	1·225
1·6	126·4911	4·	2·828	2·309	2·	1·789	1·632	1·511	1·414	1·333	1·265
1·7	130·3840	4·123	2·915	2·380	2·061	1·844	1·683	1·558	1·457	1·374	1·304
1·8	134·1641	4·243	3·	2·449	2·121	1·897	1·731	1·604	1·5	1·414	1·342
1·9	137·8405	4·359	3·082	2·517	2·179	1·949	1·779	1·648	1·541	1·453	1·378
2·	141·4214	4·472	3·162	2·582	2·236	2·	1·825	1·691	1·581	1·491	1·414
2·1	144·9138	4·583	3·240	2·646	2·291	2·049	1·871	1·732	1·620	1·528	1·449
2·2	148·3240	4·690	3·317	2·707	2·345	2·098	1·914	1·773	1·658	1·563	1·483
2·3	151·6575	4·796	3·391	2·769	2·398	2·145	1·958	1·812	1·695	1·599	1·517
2·4	154·9193	4·899	3·464	2·828	2·449	2·191	1·999	1·852	1·732	1·633	1·549
2·5	158·1139	5·	3·536	2·886	2·5	2·236	2·040	1·889	1·768	1·666	1·581
2·6	161·2452	5·099	3·606	2·943	2·549	2·280	2·081	1·927	1·803	1·699	1·612
2·7	164·3168	5·196	3·674	3·	2·598	2·324	2·121	1·964	1·837	1·732	1·643
2·8	167·3320	5·292	3·742	3·055	2·646	2·366	2·160	2·	1·871	1·764	1·673
2·9	170·2939	5·385	3·808	3·109	2·692	2·408	2·198	2·035	1·904	1·795	1·703
3·	173·2051	5·477	3·873	3·163	2·738	2·449	2·236	2·070	1·936	1·826	1·732

TABLE VII.—*continued.*

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·00016	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
3·1	176·0682	5·568	3·937	3·215	2·784	2·490	2·273	2·105	1·968	1·856	1·761
3·2	178·8854	5·657	4·	3·266	2·828	2·530	2·309	2·138	2·	1·886	1·789
3·3	181·6590	5·745	4·062	3·317	2·872	2·569	2·845	2·172	2·031	1·915	1·817
3·4	184·3909	5·831	4·123	3·367	2·915	2·608	2·382	2·204	2·061	1·944	1·844
3·5	187·0829	5·916	4·183	3·416	2·958	2·646	2·415	2·236	2·092	1·972	1·871
3·6	189·7367	6·	4·243	3·464	3·	2·683	2·449	2·267	2·121	2·	1·897
3·7	192·3538	6·083	4·301	3·512	3·041	2·720	2·483	2·299	2·150	2·028	1·924
3·8	194·9359	6·164	4·359	3·559	3·082	2·757	2·516	2·330	2·179	2·055	1·949
3·9	197·4842	6·245	4·416	3·606	3·122	2·793	2·548	2·360	2·208	2·082	1·975
4·	200·	6·325	4·472	3·651	3·162	2·828	2·581	2·390	2·236	2·108	2·
4·1	202·4846	6·403	4·528	3·696	3·202	2·864	2·613	2·421	2·264	2·134	2·025
4·2	204·9390	6·481	4·583	3·741	3·240	2·898	2·645	2·450	2·291	2·160	2·049
4·3	207·3644	6·557	4·637	3·786	3·278	2·933	2·680	2·480	2·318	2·186	2·074
4·4	209·7618	6·633	4·690	3·829	3·316	2·966	2·707	2·507	2·345	2·211	2·098
4·5	212·1320	6·708	4·743	3·873	3·354	3·	2·738	2·535	2·371	2·236	2·121
4·6	214·4761	6·782	4·796	3·916	3·391	3·033	2·769	2·564	2·398	2·261	2·145
4·7	216·7948	6·856	4·848	3·958	3·428	3·066	2·798	2·591	2·424	2·285	2·168
4·8	219·0890	6·928	4·899	4·	3·464	3·098	2·828	2·619	2·449	2·309	2·191
4·9	221·3594	7·	4·950	4·041	3·5	3·130	2·857	7·646	2·475	2·333	2·214
5·	223·6068	7·071	5·	4·082	3·535	3·162	2·886	2·672	2·5	2·357	2·236

TABLE VII.—*continued.*

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
5·1	225·8318	7·141	5·050	4·122	3·570	3·194	2·914	2·699	2.525	2·380	2·258
5·2	228·0351	7·211	5·099	4·164	3·605	3·225	2·944	2·725	2·549	2·404	2·280
5·3	230·2173	7·280	5·148	4·204	3·640	3·258	2·972	2·751	2·574	2·427	2·302
5·4	232·3790	7·348	5·196	4·242	3·674	3·286	2·999	2·777	2·598	2·449	2·324
5·5	234·5208	7·416	5·244	4·282	3·708	3·317	3·027	2·803	2·622	2·472	2·345
5·6	236·6432	7·483	5·292	4·320	3·742	3·347	3·054	2·828	2·646	2·494	2·366
5·7	238·7467	7·550	5·339	4·359	3·775	3·376	3·080	2·854	2·669	2·517	2·387
5·8	240·8319	7·616	5·385	4·397	3·808	3·406	3·109	2·878	2·692	2·539	2·408
5·9	242·8992	7·681	5·431	4·434	3·840	2·435	3·135	2·903	2·715	2·560	2·429
6·	244·9490	7·746	5·477	4·472	3·873	3·464	3·162	2·928	2·738	2·582	2·449
6·1	246·9818	7·810	5·523	4·508	3·905	3·493	3·187	2·952	2·761	2·603	2·470
6·2	248·9980	7·874	5·568	4·546	3·937	3·521	3·214	2·977	2·784	2·625	2·490
6·3	250·9980	7·937	5·612	4·583	3·968	3·550	3·240	3·	2·806	2·646	2·510
6·4	252·9822	8·	5·657	4·619	4·	3·578	3·264	3·024	2·828	2·666	2·530
6·5	254·9510	8·062	5·701	4·654	4·031	3·606	3·290	3·048	2·850	2·687	2·550
6·6	256·9047	8·124	5·745	4·690	4·062	3·633	3·316	3·071	2·872	2·708	2·570
6·7	258·8436	8·185	5·788	4·725	4·093	3·661	3·340	3·093	2·894	2·728	2·588
6·8	260·7681	8·246	5·831	4·761	4·123	3·688	3·366	3·117	2·915	2·749	2·608
6·9	262·6785	8·307	5·874	4·796	4·153	3·715	3·391	3·138	2·937	2·769	2·627
7·	264·5751	8·367	5·916	4·830	4·184	3·742	3·415	3·162	2·957	2·789	2·646

TABLE VII.—*continued.*

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
7·1	266·4583	8·426	5·958	4·865	4·213	3·768	3·439	3·185	2·979	2·809	2·665
7·2	268·3282	8·485	6·	4·898	4·242	3·795	3·463	3·207	3·	2·828	2·683
7·3	270·1851	8·544	6·042	4·933	4·272	3·821	3·483	3·229	3·021	2·848	2·702
7·4	272·0294	8·602	6·083	4·966	4·301	3·847	3·511	3·252	3·041	2·867	2·720
7·5	273·8613	8·660	6·124	5·	4·330	3·873	3·535	3·273	3·062	2·887	2·739
7·6	275·6810	8·718	6·164	5·033	4·359	3·899	3·558	3·296	3·082	2·906	2·757
7·7	277·4887	8·775	6·205	5·066	4·387	3·924	3·582	3·317	3·102	2·925	2·775
7·8	279·2848	8·832	6·245	5·099	4·416	3·950	3·605	3·339	3·122	2·944	2·793
7·9	281·0694	8·888	6·285	5·132	4·444	3·975	3·628	3·360	3·142	2·963	2·811
8·	282·8427	8·944	6·325	5·163	4·472	4·	3·650	3·380	3·162	2·981	2·828
8·1	284·6050	9·	6·364	5·196	4·5	4·025	3·674	3·400	3·182	3·	2·846
8·2	286·3564	9·055	6·403	5·229	4·527	4·050	3·697	3·423	3·201	3·018	2·863
8·3	288·0972	9·110	6·442	5·261	4·555	4·074	3·719	3·443	3·221	3·037	2·881
8.4	289·8275	9·165	6·481	5·291	4·582	4·099	3·741	3·464	3·240	3·055	2·898
8·5	291·5476	9·220	6·519	5·322	4·610	4·123	3·763	3·485	3·259	3·073	2·915
8·6	293·2576	9·274	6·557	5·354	4·637	4·147	3·785	3·505	3·278	3·091	2·933
8·7	294·9576	9·327	6·595	5·385	4·663	4·171	3·807	3·525	3·297	3·109	2·950
8·8	296·6479	9·381	6·633	5·416	4·690	4·195	3·829	3·545	3·316	3·127	2·966
8·9	298·3287	9·434	6·671	5·447	4·717	4·219	3·851	3·566	3·335	3·145	2·983
9·	300·	9·487	6·708	5·477	4·743	4·243	3·872	3·586	3·354	3·162	3·

TABLE VII.—*continued.*

Hydraulic mean radius in feet.	Tabular No. to be multiplied by $\sqrt{S}$ for other values.	Values of S.									
		·001	·0005	·00033	·00025	·0002	·000166	·000143	·000125	·000111	·0001
		Velocities of discharge in feet per second.									
9·1	301·6621	9·539	6·745	5·506	4·769	4·266	3·893	3·606	3·372	3·179	3·017
9·2	303·3150	9·592	6·782	5·537	4·796	4·290	3.915	3·625	3·391	3·197	3·033
9·3	304·9590	9·644	6·819	5·568	4·822	4·313	3.936	3·645	3·409	3·215	3·050
9·4	306·5942	9·695	6·856	5·599	4·847	4·336	3.958	3·665	3·428	3·232	3·066
9·5	308·2207	9·747	6·892	5·630	4·873	4·359	3.980	3·685	3·446	3·249	3·082
9·6	309·8387	9·798	6·928	5·658	4·899	4·382	4·	3·704	3·464	3·266	3·098
9·7	311·4482	9·849	6·964	5·686	4·924	4·405	4·020	3·723	3·482	3·283	3·114
9·8	313·0495	9·899	7·	5·714	4·949	4·427	4·039	3·741	3·5	3·299	3·130
9·9	314·6427	9·950	7·036	5·745	4·975	4·450	4·060	3·761	3·518	3·317	3·146
10	316·2278	10·	7·071	5·773	5·	4·472	4·082	3·778	3·535	3·333	3·162
15	387·2983	12·247	8·660	7·070	6·123	5·477	4·998	4·629	4·330	4·082	3·873
20	447·2136	14·142	10·	8·165	7·071	6·325	5·773	5·346	5·	4·714	4·472
25	500·	15·811	11·180	9·128	7·905	7·071	6·453	5·977	5·590	5·270	5·
30	547·7226	17·321	12·247	10·	8·660	7·746	7·070	6·546	6·123	5 773	5·477
35	591·6080	18·708	13·229	10·801	9·354	8·367	7·636	7·071	6·614	6·236	5·916
40	632·4555	20·	14·142	11·545	10·	8·944	8·162	7·559	7·071	6·666	6·325
45	670·8204	21·213	15·	12·247	10·606	9·487	8·659	8·017	7·5	7·071	6·708
50	707·1068	22·361	15·811	12·910	11·180	10.	9·127	8·456	7·905	7·454	7·071
60	774·5967	24·495	17·321	14·142	12·242	10·954	10·	9·258	8·660	8·165	7·746
70	836·6600	26·458	18·708	15·275	13·229	11·832	10·799	10·	9·354	8·819	8·367

Explanatory Examples to Table VII.

Example 1. A river has a hydraulic radius of 5·2 feet, a hydraulic slope of ·0002 and a cross section of 1000 square feet, required the discharge, assuming a frictional co-efficient of ·03.

By Table VII. the unmodified mean velocity of discharge = 3·225 feet per second, and by Part 3 of Table XII. the value of c the co-efficient suitable to this radius and slope is ·668, hence the true discharge = $c \times A \times Q$ = ·66 × 1000 × 3·225 = 2128 cubic feet per second.

Example 2. Suppose the river mentioned in the last example to have a hydraulic slope of ·0015, the remaining data being as before, required the discharge.

In this case the inclination not being one of those given at the heads of columns, make use of the tabular number corresponding to the hydraulic radius, which is 228·03, and multiplying it by $\sqrt{\cdot 0015}$, an unmodified mean velocity of discharge 8·87 feet per second is obtained. Taking the suitable co-efficient c from Part 3, Table XII., the true discharge = $c \times A \times V$ = ·65 × 1000 × 8·87 = 5765 cubic feet per second.

Example 3. A canal is to have a cross section of 250 square feet, a hydraulic radius of 4 feet, and must discharge when in perfect order and regimen 500 cubic feet per second, what is the hydraulic slope necessary, and what will its discharge be when it wears itself into a state resembling a natural channel, if we assume the other data to remain the same?

From an inspection of Table VII. and the table of co-efficients for artificial channels, it appears that for the given radius a co-efficient of ·753 and a slope of ·00018 would nearly satisfy the conditions: assuming ·753, the mean velocity becomes 2·65 and the slope ·000175. The discharge for a natural channel would require the co-efficient ·632, and would = ·632 × 250 × 200 × $\sqrt{\cdot 000175}$ = 418 cubic feet per second.

TABLE VIII.

For full cylindrical tubes—Pipes, Sewers, &c.

PART 1.—Discharges in cubic feet per second.

$$Q = c \times 39{\cdot}27 \left(Sd^5\right)^{\frac{1}{2}}$$

PART 2.—Diameters in feet and decimals.

$$d = \frac{1}{c^{\frac{2}{5}}} \times {\cdot}23 \left(\frac{Q^2}{S}\right)^{\frac{1}{5}}$$

PART 3.—Heads for a length of 100 feet, in feet.

$$H = \frac{1}{c^2} \times {\cdot}0648 \frac{Q^2}{d^5}$$

being values of the corresponding formulæ, when c = 1.

N.B.—For more correct results, apply the values of the co-efficient (*c*) given in Part 3, Table XII., in every case, using the table of useful numbers, Part 7, Table XII., for powers and roots.

The tabular numbers extend the use of the tables to any slope.

Some explanatory examples follow this table.

TABLE VIII.—Part 1.

Discharges through full cylindrical tubes, Pipes, Culverts, &c.

For diameters in feet.	For slopes of one in 100	150	200	300	400	500	1000	Tabular No. to be multiplied by $\sqrt{s}$ for other slopes.
	Discharges in cubic feet per second.							
(1″) ·083	·008	·006	·006	·005	·004	·004	·003	·079
(2″) ·166	·04	·04	·03	·03	·02	·02	·01	·445
(3″) ·25	·12	·10	·09	·07	·06	·05	·04	1·227
(4″) ·33	·25	·21	·18	·15	·13	·11	·08	2·519
(5″) ·416	·44	·36	·31	·25	·22	·20	·14	4·401
(6″) ·5	·69	·57	·49	·40	·35	·31	·22	6·939
(7″) ·583	1·02	·83	·72	·59	·51	·46	·32	10·206
(8″) ·66	1·43	1·16	1·01	·82	·71	·64	·45	14·251
(9″) ·75	1·91	1·56	1·35	1·10	·97	·86	·61	19·128
(10″) ·83	2·49	2·03	1·76	1·44	1·25	1·11	·79	24·895
(11″) ·916	3·16	2·58	2·23	1·82	1·58	1·41	1·00	31·594
(12″) 1·00	3·93	3·28	2·78	2·27	1·96	1·76	1·24	39·27
1·25	6·86	5·60	4·85	3.96	3·43	3·07	2·16	68·601
1·5	10·82	8·82	7·65	6·25	5·41	4·84	3·42	108·216
1·75	15·91	12·99	11·25	9·18	7·95	7·11	5·03	159·095
2·	22·21	18·14	15·71	12·83	11·11	9·93	7·02	222·146
2·25	29·82	24·35	21·08	17·22	14·91	13·34	9·43	298·505
2·5	38·81	31·69	27·44	22·41	19·40	17·35	12·27	388·078
2·75	49·25	40·22	34·82	28·43	24·62	22·02	15·57	492·489
3·	61·21	49·99	43·28	35·34	30·61	27·37	19·35	612·105
3·25	74·77	61·04	52·87	43·18	37·38	33·44	23·64	747·744
3·5	89·99	73·49	63·63	51·96	44·99	40·25	28·46	899·990
3·75	106·94	87·33	75·61	61·74	53·46	47·82	33·81	1069·397
4·	125·66	102·63	88·84	72·55	62·83	56·20	39·73	1256·640
4·25	146·23	119·42	103·38	84·32	73·11	65·39	46·24	1462·262
4·5	168·69	137·76	119·26	97·39	84·34	75·44	53·34	1686·886
4·75	193·10	157·70	136·52	111·48	96·55	86·36	61·06	1931·028
5·	219·54	179·26	155·24	126·75	109·77	99·18	69·43	2195·436
5·5	278·61	227·48	197·00	160·85	139·30	124·60	88·10	2786·060
6·	346·31	282·76	244·88	199·94	173·16	154·88	109·51	3463·130
6·5	423·03	345·40	299·13	244·23	211·51	189·18	133·77	4230·262
7·	509·13	415·70	360·01	293·95	254·57	227·69	161·00	5091·322

TABLE VIII.—Part 2.

Diameters of full Pipes of small discharge and high inclination.

Discharges in cubic feet per second.	For slopes of one in 100	150	200	300	400	500	1000	Tabular No. to be multiplied by $(\frac{1}{s})^{\frac{1}{5}}$ for other slopes.
	Diameters in feet.							
·1	·23	·25	·26	·29	·30	·32	·36	·0916
·2	·30	·33	·35	·38	·40	·42	·48	·1208
·3	·36	·39	·41	·44	·47	·49	·57	·1421
·4	·40	·43	·46	·50	·53	·55	·63	·1594
·5	·44	·47	·50	·55	·58	·60	·69	·1743
·6	·47	·51	·54	·59	·62	·65	·75	·1875
·7	·50	·54	·58	·62	·66	·69	·79	·1994
·8	·53	·57	·61	·66	·70	·73	·84	·2104
·9	·56	·60	·64	·69	·73	·77	·88	·2215
1·	·58	·63	·66	·72	·76	·80	·92	·2300
1·1	·60	·65	·69	·75	·79	·83	·95	·2385
1·2	·62	·67	·71	·77	·82	·86	·99	·2474
1·3	·64	·70	·74	·80	·85	·89	1·02	·2556
1·4	·66	·72	·76	·82	·87	·91	1·05	·2631
1·5	·68	·74	·78	·85	·90	·94	1·08	·2705
1·6	·70	·76	·80	·87	·92	·96	1·11	·2776
1·7	·71	·77	·82	·89	·94	·99	1·13	·2844
1·8	·73	·79	·84	·91	·96	1·01	1·16	·2910
1·9	·75	·81	·86	·93	·99	1·03	1·18	·2973
2·0	·76	·83	·88	·95	1·01	1·05	1·21	·3035
2·1	·78	·84	·89	·97	1·03	1·07	1·23	·3095
2·2	·79	·86	·91	·99	1·04	1·09	1·26	·3153
2·3	·81	·87	·93	1·01	1·06	1·11	1·28	·3209
2·4	·82	·89	·94	1·02	1·08	1·13	1·30	·3265
2·5	·83	·90	·96	1·04	1·10	1·15	1·32	·3318
2·6	·85	·92	·97	1·05	1·12	1·17	1·34	·3368
2·7	·86	·93	·99	1·07	1·13	1·19	1·36	·3422
2·8	·87	·95	1·00	1·09	1·15	1·20	1·38	·3472
2·9	·88	·96	1·02	1·10	1·17	1·22	1·40	·3521
3·0	·90	·97	1·03	1·12	1·18	1·24	1·42	·3569

For special cases modify the discharge by a co-efficient (c) before *applying it to the* table, to find the diameter.

TABLE VIII.—Part 2—*continued.*

Diameters of full cylindrical Sewers, Drains of large discharge and low inclination.

Discharges in cubic feet per second.	For slopes of one in 500	1000	1500	2000	2500	3000	4000	Tabular No. to be multiplied by $\left(\frac{1}{s}\right)^{\frac{1}{5}}$ for other slopes.
	Diameters in feet.							
1	·80	·92	·99	1·05	1·10	1·14	1·21	·23
2	1·05	1·21	1·31	1·39	1·45	1·51	1·59	·30348
3	1·24	1·42	1·54	1·63	1·71	1·77	1·88	·35692
4	1·39	1·59	1·73	1·83	1 91	1·99	2·10	·40045
5	1·52	1·74	1·89	2·00	2·09	2·17	2·30	·43780
6	1·63	1·87	2·03	2·15	2·25	2·34	2·47	·47096
7	1·74	1·99	2·16	2·29	2·40	2·48	2·63	·50092
8	1·83	2·10	2·28	2·42	2·53	2·62	2·78	·52840
9	1·92	2·21	2·39	2·53	2·65	2·75	2·91	·55389
10	2·00	2.30	2·49	2·64	2·76	2·87	3·04	·57773
11	2·08	2·39	2·59	2·74	2·87	2·98	3·15	·60018
12	2·15	2·47	2·68	2·84	2·97	3·08	3·27	·62144
13	2·22	2·55	2·76	2·93	3·07	3·18	3·37	·64166
14	2·29	2·63	2·85	3·02	3·16	3·28	3·47	·66096
15	2·36	2·71	2·93	3·11	3·25	3·37	3·57	·67946
16	2·42	2·78	3·01	3·19	3·32	3·46	3·66	·69723
17	2·48	2·84	3·08	3·27	3·42	3·54	3·75	·71434
18	2·53	2.91	3·16	3·34	3·50	3·63	3·84	·73086
19	2·59	2·97	3·22	3·42	3·57	3·70	3·92	·74684
20	2·64	3·03	3·29	3·49	3·65	3·78	4·00	·76232
30	3·11	3·57	3·87	4·10	4·29	4·45	4·71	·89655
40	3·49	4·00	4·34	4·60	4·81	4·99	5·28	1·0059
50	3·81	4·38	4·75	5·03	5·26	5·45	5·78	1·0998
60	4·10	4·71	5·11	5·41	5·66	5·87	6·21	1·1830
70	4·36	5·01	5·43	5·75	6·02	6·24	6·61	1·2582
80	4·60	5·28	5·73	6·07	6·35	6·58	6·97	1·3273
90	4·82	5·54	6·00	6·36	6·65	6·90	7·31	1·3913
100	5·03	5·78	6·26	6·64	6·94	7·20	7·62	1·4512
200	6·64	7·62	8·27	8·76	9·16	9·50	10·06	1·9149
300	7·81	8·97	9·72	10·30	10·77	11·16	11·83	2·2520

For special cases, modify the discharge by a co-efficient (*c*) before applying it to the table, to find the diameter.

TABLE VIII.—Part 3.

Small Pipes. Heads for a length of 100 feet.

For discharges in cubic feet per second.	For diameters in feet. ·083 (1″)	·166 (2″)	·25 (3″)	·333 (4″)	·416 (5″)	Tabular number to be divided by d^5 for other diameters.
	Head of water in feet.					
·1	161	5.04	·664	·157	·0516	·000648
·2	645	20·16	2·654	·630	·2064	·002592
·3	1451	45·35	5·972	1·417	·4644	·005832
·4	2580	80·62	10·617	2·525	·8256	·010368
·5	4031	125·97	16·589	3·937	1·2899	·0162
·6	5804	180·56	23·888	5·669	1·8575	·023328
·7	7900	246·90	32·514	7·716	2·5283	·031752
·8	10318	322·48	42·467	10·078	3·3023	·041472
·9	13061	408·15	53·748	12·754	4·1794	·052488
1·0	16124	503·89	66·355	15·746	5·1598	·0648
1·1	19510	609·70	80·290	19·053	6·2433	·078408
1·2	23219	725·59	95·551	22·675	7·4301	·093312
1·3	27250	851·56	112·140	26·611	8·7200	·109512
1·4	31604	987·60	130·056	30·863	10·1132	·127008
1·5	36280	1133·74	149·299	35·429	11·6095	·1458
1·6	41278	1289·94	169·869	40·311	13·2090	·165888
1·7	46599	1456·22	191·767	45·507	14·9118	·187272
1·8	52243	1632·59	214·992	51·018	16·7177	·209952
1·9	58209	1819·02	239·542	56·844	18·6268	·233928
2·0	64497	2015·54	265·421	62·986	20·6391	·2592
2·1	71108	2222·13	292·627	69·442	22·7546	·285768
2·2	78042	2438·80	321·160	76·212	24·9733	·313632
2·3	85298	2665·55	351·020	83·299	27·2952	·342792
2·4	92876	2902·36	382·206	90·699	29·7203	·373248
2·5	100777	3149·28	414·720	98·415	32·2487	·4050
2·6	109000	3406·26	448·561	106·446	34·8801	·438048
2·7	117546	3673·32	483·730	114·791	37·6147	·472392
2·8	126415	3950·46	520·225	123·452	40·4527	·508032
2·9	135605	4237·67	558·048	132·427	43·3937	·544968
3·0	145119	4534·96	597·197	141·717	46·4380	·5832

For special cases modify the discharge by a co-efficient (c) before *applying it*, to find the head necessary.

TABLE VIII.—PART 3—*continued.*

Pipes. Head for a length of 100 feet.

For discharges in cubic feet per second.	For diameters in feet. ·5 (6″)	·583 (7″)	·666 (8″)	·75 (9″)	·833 (10″)	Tabular number to be divided by d^5 for other diameters.
	Head of water in feet and decimals.					
·1	·0207	·0096	·0049	·0027	·0016	·000648
·2	·0829	·0384	·0197	·0107	·0064	·002592
·3	·1866	·0863	·0443	·0246	·0145	·005832
·4	·3318	·1535	·0787	·0437	·0258	·010368
·5	·5184	·2398	·1230	·0683	·0403	·0162
·6	·7465	·3454	·1772	·0989	·0580	·023328
·7	1·0163	·4701	·2411	·1338	·0790	·031752
·8	1·3271	·6140	·3149	·1748	·1032	·041472
·9	1·6796	·7753	·3995	·2212	·1306	·052488
1·0	2·0736	·9594	·4921	·2731	·1612	·0648
1·1	2·5091	1·1608	·5954	·3304	·1951	·078408
1·2	2·9860	1·3815	·7086	·3932	·2322	·093312
1·3	3·5044	1·6213	·8316	·4615	·2725	·109512
1·4	4·0643	1·8804	·9645	·5352	·3160	·127008
1·5	4·6656	2·1586	1·1072	·6144	·3628	·1458
1·6	5·3084	2·4560	1·2597	·6991	·4128	·165888
1·7	5·9927	2·7726	1·4221	·7892	·4660	·187272
1·8	6·7185	3·1084	1·5943	·8847	·5224	·209952
1·9	7·4857	3·4633	1·7764	·9858	·5821	·233928
2·0	8·2944	3·8375	1·9683	1·0674	·6450	·2592
2·1	9·1446	4·2309	2·1701	1·2042	·7111	·285768
2·2	10·0362	4·6377	2·3816	1·3216	·7804	·313632
2·3	10·9693	5·0751	2·6031	1·4445	·8530	·342792
2·4	11·9439	5·5260	2·8343	1·5729	·9288	·373248
2·5	12·9600	5·9961	3·0755	1·7067	1·0078	·4050
2·6	14·0175	6·4854	3·3264	1·8459	1·0900	·438048
2·7	15·1165	6·9939	3·5872	1·9906	1·1755	·472392
2·8	16·2570	7·5215	3·8579	2·1408	1·2642	·508032
2·9	17·4390	8·0683	4·1383	2·2965	1·3561	·544968
3·0	18·6624	8·6344	4·4287	2·4576	1·4512	·5832

For special cases modify the discharge by a co-efficient (c) before applying it, to find the head necessary.

TABLE VIII.—Part 3—*continued.*

Large Pipes. Head for a length of 100 feet.

For discharges in cubic feet per second.	For diameters in feet. 1·	1·5	2·	2·5	3·0	Tabular number to be divided by d^5 for other diameters.
	Head of water in feet.					
1	·0648	·0085	·0020	·0007	·0003	·0648
2	·2592	·0341	·0081	·0027	·0011	·2592
3	·5832	·0768	·0182	·0060	·0024	·5832
4	1·0368	·1365	·0324	·0106	·0043	1·0368
5	1·62	·2133	·0506	·0166	·0067	1·62
6	2·3328	·3072	·0729	·0239	·0096	2·3328
7	3·1752	·4181	·0992	·0325	·0131	3·1752
8	4·1472	·5461	·1296	·0425	·0167	4·1472
9	5·2488	·6912	·1640	·0537	·0216	5·2488
10	6·48	·8533	·2025	·0664	·0267	6·48
11	7·8408	1·0325	·2450	·0803	·0323	7·8408
12	9·3312	1·2288	·2916	·0956	·0384	9·3312
13	10·9512	1·4421	·3422	·1121	·0451	10·9512
14	12·7008	1·6725	·3969	·1301	·0523	12·7008
15	14·58	1·92	·4556	·1493	·0600	14·58
16	16·5888	2·1845	·5184	·1699	·0667	16·5888
17	18·7272	2·4661	·5852	·1918	·0771	18·7272
18	20·9952	2·7648	·6561	·2150	·0864	20·9952
19	23·3928	3·0805	·7310	·2395	·0963	23·3928
20	25·92	3·4133	·81	·2654	·1067	25·92
21	28·5768	3·7632	·8930	·2926	·1176	28·5768
22	31·3632	4·1301	·9801	·3212	·1291	31·3632
23	34·2792	4 5141	1·0712	·3510	·1411	34·2792
24	37·3248	4·9152	1·1664	·3822	·1536	37·3248
25	40·50	5·3333	1·2656	·4147	·1667	40·50
26	43·8048	5·7685	1·3689	·4486	·1803	43·8048
27	47·2392	6·2208	1·4762	·4837	·1944	47·2392
28	50·8032	6·6901	1·5876	·5202	·2091	50·8032
29	54·4968	7·1765	1·7030	·5580	·2243	54·4968
30	58·32	7·68	1·8225	·5972	·2400	58·32

For special cases, modify the discharge by a co-efficient (c) before applying it, to find the head necessary.

TABLE VIII.—PART 3—*continued.*

Cylindrical Sewers or Tunnels. Head for a length of 100 feet.

For discharges in cubic feet per second.	For diameters in feet. 3	4	5	6	7	Tabular numbers to be divided by d^5 for other diameters.
	Head of water in feet.					
1	·0003	·00006	·00002	·000008	·000004	·06
2	·0011	·00025	·00008	·000033	·000015	·26
3	·0024	·00057	.00018	·000075	·000035	·58
4	·0043	·00101	·00033	·000133	·000062	1·04
5	·0067	·00158	·00052	·000208	·000096	1·62
6	·0096	·00228	·00074	·000300	·000139	2·33
7	·0131	·00310	·00102	·000408	·000189	3·18
8	·0167	·00405	·00133	·000533	·000247	4·15
9	·0216	·00513	·00168	·000675	·000312	5·25
10	·0267	·00633	.00207	·000833	·000386	6·48
15	·0600	·01424	·00466	·001875	·000868	14·58
20	·1067	·02531	·00829	·003333	·001542	25·92
25	·1667	·03955	·01296	·005208	·002410	40·50
30	·2400	·05695	·01866	·007500	·003470	58·32
35	·3267	·07752	·02540	·010208	·004723	79·38
40	·4267	·10132	·03318	·013333	·006169	103·68
45	·5400	·12815	·04199	·016875	·007875	131·22
50	·6667	·15823	·05184	·020833	·009639	162·00
55	·8067	·19143	·06273	·025208	·011663	196·02
60	·9600	·22781	·07465	·030000	·013880	233·28
65	1·1267	·26736	·08761	·035208	·016289	273.78
70	1·3067	·31008	·10161	·030833	·018892	317·52
75	1·5000	·35596	·11664	·046875	·021687	364·50
80	1·6678	·40500	·13271	·053333	·024675	414·72
85	1·9267	·45721	·14982	·060208	·027856	468·18
90	2·1600	·51258	·16796	·067500	·031230	524·88
95	2·4067	·57112	·18714	·075208	·034796	584·82
100	2·6667	·63281	·20736	·083333	·038555	648·
200	10·6667	2·53120	1·82944	·333338	·154222	2592·
300	24·	5·69530	1·86624	.750000	·346998	5832·

For special cases, modify the discharge by a co-efficient (c) before applying it, to find the head necessary.

Explanatory Examples to Table VIII.

Example 1. What is the discharge of an enamelled 3-inch pipe having a hydraulic slope of 1 in 400; and what would be its least full discharge when old, irrespectively of sectional obstruction?

By Table VIII., Part 1, the tabular discharge is ·06 cubic feet per second; and by the Table of co-efficients (Table XII., Part 3), for very smooth surfaces, including smooth plaster, and enamelled or glazed pipes, the co-efficient c for a pipe having this slope and a hydraulic radius, which for cylindrical pipes running full is one-fourth of the diameter, is ·84, the discharge when new is $= \cdot 84 \times \cdot 06 = \cdot 05$ cubic feet per second.

If preferred in any other unit, refer to Table XI., Part 2, by inspecting which we find it $= 18$ gallons per minute.

When the pipe is old its surface will not be rougher than that of ordinary metal, and taking the co-efficient for metal with this slope and radius to be ·61, the least discharge is $= \cdot 61 \times \cdot 06 = \cdot 037$ cubic feet per second, or 14 gallons per minute.

Example 2. A masonry culvert has a diameter of 42 inches, and a slope of 1 in 200, what is its discharge when running full?

By Part 1, Table VIII., the tabular discharge is 63·63 cubic feet per second, and the co-efficient for this slope and a hydraulic radius of ·875 feet will according to Table XII. be 1·10; hence the actual discharge will be $1 \cdot 10 \times 63 \cdot 63 = 70$ cubic feet per second.

Example 3. What must be the diameter of a cast iron pipe to discharge 20 cubic feet per second with a slope of one in 500?

By Part 2, Table VIII., the tabular diameter will be 2·64 feet and the hydraulic radius ·66 feet; turning to the table of co-efficients (Table XII., Part 3), we take $c = 1 \cdot 03$: and assuming a modified discharge $\frac{Q}{c} = 19 \cdot 4$, and referring again to Part 2, Table VIII., we obtain a true diameter $= 2 \cdot 62$ feet.

Example 4. What should be the dimensions of an ovoidal brickwork sewer to discharge 50 cubic feet per second with a slope of 1 in 1,000, the sewer flowing two-thirds full?

The co-efficient to modify the discharge through cylindrical into

that for ovoidal sewers of the usual type running two-thirds full is generally assumed to be $= \frac{35}{39 \cdot 27} = \cdot 89$; hence the first modification of discharge will be $\cdot 89 \times 50 = 44 \cdot 5$: using this and referring to Part 2, Table VIII., we get a first approximation to a diameter of 4·19 feet. Secondly, referring to the table of co-efficients, Table XII., we obtain a co-efficient c, corresponding to a hydraulic radius of 1·05 and a slope of ·001, $= 1 \cdot 13$; and modifying the discharge a second time it becomes $= \frac{44 \cdot 5}{1 \cdot 13} = 39 \cdot 4$ giving, according to Part 2, Table VIII., 3·97 feet for the diameter of a cylindrical sewer. Hence the dimensions for the corresponding ovoidal sewer will be

d	diameter of top circle	$= 3 \cdot 97$
$\frac{d}{2}$	diameter of bottom circle	$= 1 \cdot 98$
$\frac{3d}{2}$	radius of each side circle	$= 5 \cdot 96$
$\frac{3d}{2}$	depth of sewer	$= 5 \cdot 96$

Example 5. A series of enamelled pipes has a total head of 30 feet, and consists of 3600 feet of 8-inch pipe, 2100 feet of 6-inch, and 600 feet of 5-inch; required the discharge and head necessary for each pipe.

Assume any discharge as 1 cubic foot per second, and obtaining the separate tabular heads due to it, divide the total head in the same proportion.

$\cdot 4921 \times 36 = 17 \cdot 72$	$17 \cdot 72 \times 30 \div 92 = 5 \cdot 77$
$2 \cdot 0736 \times 21 = 43 \cdot 55$	$43 \cdot 55 \times 30 \div 92 = 14 \cdot 15$
$5 \cdot 1598 \times 6 = 30 \cdot 95$	$30 \cdot 95 \times 30 \div 92 = 10 \cdot 08$
Total $= 92 \cdot 22$	Total $= 30 \cdot$

And modifying these by the squares of the suitable co-efficients, obtain actual heads for a first approximation.

$5 \cdot 77 \div (\cdot 95)^2 = 6 \cdot 41$	$6 \cdot 41 \times 30 \div 39 \cdot 22 = 4 \cdot 90$
$14 \cdot 15 \div (\cdot 87)^2 = 18 \cdot 62$	$18 \cdot 62 \times 30 \div 39 \cdot 22 = 14 \cdot 24$
$10 \cdot 08 \div (\cdot 84)^2 = 14 \cdot 19$	$14 \cdot 19 \times 30 \div 39 \cdot 22 = 10 \cdot 86$
Total $= 39 \cdot 22$	Total $= 30 \cdot$

and the discharge $= \frac{1 \times \sqrt{30}}{\sqrt{92}} = \cdot 57$ cubic feet per second $= 213$ gallons per minute.

EXAMPLE 6. A discharge of 300 gallons per minute is required through a series of ordinary iron pipes composed of 800 yards of 7-inch, 300 yards of 6-inch, and 100 yards of 5-inch; what is the head required for each pipe?

By Tables of equivalents (Part 2, Table XI.), 300 gals. per minute = ·8 cubic feet per second, and the corresponding tabular heads in Part 3, Table VIII., can be taken as a first approximation, and modifying these by the squares of the suitable co-efficients given in Table XII. we get the true heads thus:—

	Length.	Head.		True Heads.
7 inch	·6140 ×	24 =	14·74	14·74 ÷ $(·66)^2$ = 33·50
6 inch	1·3271 ×	9 =	11·94	11·94 ÷ $(·63)^2$ = 29·85
5 inch	3·3023 ×	3 =	9·91	9·91 ÷ $(·61)^2$ = 26·78
			36·59 feet.	Total 90·13 feet.

TABLE IX.

Gives velocities of discharge in feet per second for sluices, and orifices, due to various heads for certain co-efficients, also theoretical velocities to which any co-efficient may be applied; being an application of the formula

$$V = m \times 8{\cdot}025 \sqrt{H}.$$

where for orifices H = depth of centre of motion of orifice.

The same table also applies to overfalls, weirs, and notches, but in this case using the same general formula, H is the depth from still water to sill-level, and the velocities given in the table must be reduced by one-third to obtain velocities of discharge for all sorts of overfalls.

For values of (m) the co-efficient, see Parts 5 and 6, Table XII.

This table can also be used for the converse purpose.

TABLE IX.

Head in feet.	CO-EFFICIENTS.					
	For natural velocity.	For narrow bridge-openings.	For velocity of approach.	For special weirs.	For special orifices.	For bound-[illegible] [illegible].
	1·	·9	·8	·7	·6	·5
	Velocities of Discharge.					
·01	·803	·722	·642	·562	·482	·401
·02	1·135	1·021	·908	·794	·681	·567
·03	1·390	1·251	1·112	·973	·834	·695
·04	1·605	1·445	1·284	1·123	·963	·803
·05	1·794	1·615	1·435	1·256	1·076	·897
·06	1·966	1·769	1·573	1·376	1·180	·983
·07	2·123	1·911	1·698	1·486	1·274	1·062
·08	2·270	2·043	1·816	1·589	1·362	1·135
·09	2·408	2·167	1·926	1·686	1·445	1·204
·1	2·538	2·284	2·030	1·777	1·523	1·269
·2	3·589	3·230	2·871	2·512	2·153	1·794
·3	4·395	3·956	3·516	3·078	2·637	2·198
·4	5·075	4·568	4·060	3·553	3·045	2·538
·5	5·675	5·108	4·540	3·973	3·405	2·837
·6	6.216	5·594	4·973	4·351	3·730	3·108
·7	6·714	6·043	5·371	4·700	4·028	3·352
·8	7·178	6·460	5·742	5·025	4·307	3·589
·9	7·613	6·852	6·090	5·329	4·568	3·807
1	8·025	7·223	6·420	5·618	4·815	4·013

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.

Head in feet.	CO-EFFICIENTS.					
	For wide bridge-openings.	For lock sluices.	For special weirs.	For weirs generally.	For orifices generally.	For special weirs.
	·96	·84	·727	·666	·62	·55
	Velocities of Discharge.					
·01	·770	·674	·584	·535	·498	·441
·02	1·089	·953	·825	·756	·704	·624
·03	1·334	1·168	1·011	·926	·862	·765
·04	1·541	1·348	1·167	1·069	·995	·883
·05	1·722	1·507	1·304	1·185	1·112	·987
·06	1·887	1·651	1·429	1·309	1·219	1·081
·07	2·038	1·783	1·543	1·414	1·316	1·169
·08	2·179	1·907	1·650	1·512	1·407	1·249
·09	2·311	2·023	1·751	1·604	1·493	1·324
·1	2·436	2·132	1·845	1·690	1·574	1·396
·2	3·445	3·014	2·609	2·390	2·225	1·973
·3	4·219	3·694	3·195	2·927	2·725	2·418
·4	4·872	4·264	3·689	3·380	3·147	2·792
·5	5·448	4·768	4·126	3·780	3·519	3·121
·6	5·968	5·221	4·519	4·140	3·854	3·419
·7	6·445	5·640	4·881	4·471	4·163	3·687
·8	6·890	6·030	5·218	4·781	4·450	3·948
·9	7·308	6·395	5·535	5·070	4·720	4·187
1	7·704	6·742	5·834	5·345	4·976	4·414

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For natural velocity.	For narrow openings of bridges.	For velocity of approach.	For special weirs.	For special orifices.	For broad-crested dams.
	1	·9	·8	·7	·6	·5
	Velocities of Discharge.					
1·	8·0250	7·223	6·420	5·618	4·815	4·013
1·25	8·9722	8·075	7·178	6·281	5·383	4·486
1·5	9·8286	8·846	7·863	6·880	5·897	4·915
1·75	10·6161	9·554	8·493	7·431	6·370	5·308
2·	11·3491	10·214	9·079	7·944	6·809	5·675
2·25	12·0375	10·834	9·630	8·426	7·223	6·019
2·5	12·6886	11·420	10·151	8·882	7·613	6·345
2·75	13·3079	11·977	10·646	9·316	7·985	6·654
3·	13·8997	12·510	11·120	9·730	8·340	6·950
3·25	14·4678	13·020	11·574	10·127	8·680	7·234
3·5	15·0134	13·512	12·010	10·509	9·008	7·507
3·75	15·5403	13·986	12·432	10·878	9·324	7·770
4·	16·0500	14 445	12·840	11·235	9·630	8·025
4·25	16·5439	14·890	13·235	11·581	9·926	8·272
4·5	17·0235	15·322	13·619	11·916	10·214	8·512
4·75	17·4901	15·741	13·992	12·243	10·494	8·745
5·	17·9444	16·150	14·355	12·561	10 767	8·972
5·25	18·3876	16·549	14·710	12·871	11·033	9·194
5·5	18 8203	16·938	15·056	13·174	11·292	9·410
5·75	19·2433	17·319	15 395	13·470	11·546	9·622
6·	19·6572	17·691	15·726	13 760	11 794	9.829
6·25	20·0625	18·057	16 050	14·044	12·038	10·032
6·5	20·4598	18·414	16·368	14·322	12·276	10·230
6·75	20·8496	18·765	16·680	14·595	12·510	10·425
7·	21·2322	19·109	16·986	14·863	12·739	10·616
7·25	21·6079	19·447	17·286	15·126	12·965	10·804
7·5	21·9774	19·779	17·582	15·384	13·186	10·989
7·75	22 3406	20·107	17·873	15·638	13·404	11·171
8·	22·6981	20·428	18·158	15·889	13·619	11·349

N.B.—**For overfalls, reduce the tabular velocity by one-third.**

TABLE IX.—*continued.*

Head in feet.	For wide bridge-openings. ·96	For lock sluices. ·84	For special weirs. ·727	For weirs generally. ·666	For orifices generally. ·62	For special orifices. ·55
	CO-EFFICIENTS.					
	Velocities of Discharge.					
1	7·704	6·741	5·836	5·345	4·975	4·413
1·25	8·614	7·537	6·525	5·976	5·562	4·934
1·50	9·436	8 256	7·147	6·546	6·109	5·420
1·75	10·192	8·918	7·720	7·071	6·582	5·839
2	10·895	9·533	8·253	7·558	6·936	6·241
2·25	11·556	10·112	8·754	8·017	7·461	6·621
2·50	12·181	10·659	9·227	8·451	7·867	6·978
2·75	12·776	11 179	9·678	8·863	8 251	7·319
3	13·344	11·676	10·108	9·257	8·618	7·645
3·25	13 889	12·153	10·521	9·635	8·825	7·957
3·50	14·413	12·612	10·918	9·999	9·308	8·258
3·75	14·919	13·054	11·301	10·350	9·635	8·547
4	15·408	13·482	11·672	10·689	9·951	8·827
4·25	15·882	13·897	12·027	11·018	10·257	9·099
4·50	16·343	14·300	12·380	11·338	10·554	9·363
4·75	16·800	14 695	12·718	11·651	10·846	9·622
5	17·227	15·074	13·049	11·952	11·121	9·865
5 25	17·652	15·446	13·372	12·247	11·400	10·113
5·50	18·068	15·809	13·686	12·534	11·669	10·351
5·75	18·474	16·165	13·994	12·817	11·931	10 584
6	18·871	16·512	14·295	13·092	12·188	10·812
6·25	19·260	16·853	14·590	13·362	12·439	11·034
6·50	19·642	17·187	14·879	13·627	12·685	11·253
6·75	20·016	17·514	15·162	13·886	12·927	11·467
7	20·383	17·835	15·440	14·141	13·164	11·688
7·25	20·744	18·151	15·714	14·391	13 402	11·889
7·50	21·099	18·461	15·982	14·637	13·626	12·082
7·75	21 447	18·767	16·246	14·879	13·851	12·287
8	21·791	19·067	16·506	15·117	14·073	12·484

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For natural velocity.	For narrow bridge-openings.	For velocity of approach.	For special weirs.	For special orifices.	For broad-crested dams.
	1	·9	·8	·7	·6	·5
	Velocities of Discharge.					
8·25	23·051	20·746	18·441	16·135	13·831	11·525
8·50	23·397	21·057	18·717	16·377	14·032	11·698
8·75	23·739	21·365	18·992	16·617	14·243	11·869
9	24·076	21·668	19·261	16·853	14·445	12·038
9·25	24·408	21·966	19·526	17·085	14·645	12·204
9·50	24·735	22·261	19·788	17·316	14·841	12·367
9·75	25·059	22·553	20·047	17·541	15·035	12·529
10	25·378	22·840	20·302	17·764	15·227	12·689
10·5	26·005	23·404	20·804	18·203	15·603	13·002
11	26·617	23·955	21·293	18·631	15·970	13·308
11·5	27·215	24·493	21 772	19·050	16·329	13·607
12	27·800	25·020	22·240	19·460	16·680	13·900
12·5	28·373	25·535	22·698	19·861	17·024	14·186
13	28·935	26·041	23·148	20·254	17·361	14·467
13·5	29·486	26·545	23·596	20·646	17·697	14 747
14	30·027	27·024	24·021	21·019	18·016	15·013
14·5	30·559	27·503	24·447	21·391	18·335	15·279
15	31·081	27·973	24·864	21·756	18·648	15·540
15·5	31·594	28·434	25·275	22·115	18·956	15·797
16	32·101	28·891	25·681	22·470	19·261	16·050
16·5	32·598	29·338	26·078	22·818	19·555	16·299
17	33·089	29·780	26·471	23·162	19·853	16·544
17·5	33·572	30·214	26·857	23·500	20·143	16·786
18	34·048	30·643	27·238	23·833	20·429	17·024
18·5	34·518	31·066	27·614	24·162	20·711	17·259
19	34·981	31·483	27·985	24·486	20·988	17·490
19·5	35·438	31·894	28·350	24·806	21·283	17·719
20	35·889	32.300	29·711	25·122	21·533	17·944

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For wide bridge-openings.	For lock sluices.	For special weirs.	For weirs generally.	For orifices generally.	For special weirs.
	·96	·84	·727	·366	·62	·55
	Velocities of Discharge.					
8·25	22·129	19·362	16·762	15·352	14·292	12·677
8·50	22·461	19·654	17·014	15·582	14·506	12·867
8·75	22·789	19·941	17·263	15·810	14·718	13·056
9	23·112	20·223	17·508	16·034	14·927	13·242
9·25	23·431	20·502	17·749	16·256	15·133	13·424
9·50	23·746	20·778	17·987	16·473	15·336	13·604
9·75	24·056	28·049	88·223	16·689	15·536	13·782
10	24·363	21·317	18·455	16·902	15·734	13·958
10·5	24·964	21·844	18·910	17·112	16·123	14·302
11	25·552	22·358	19·355	17·727	16·502	14·639
11·5	26·126	22·860	19·791	18·125	16·873	14·968
12	26·688	23·352	20·216	18·515	17·236	15·290
12·5	27·238	23·834	20·633	18·897	17·591	15·605
13	27·778	24·306	21·042	19·271	17·940	15·914
13·5	28·307	24·769	21·442	19·637	18·287	16·222
14	28·826	25·223	21·836	19·998	18·617	16·514
14·5	29·337	25·670	22·222	20·352	18·946	16·807
15	29·838	26·108	22·602	20·700	19·270	17·094
15·5	30·331	26·540	22·976	21 042	19·588	17·377
16	30·817	26·965	23·344	21·379	19·903	17·655
16·5	31·294	27·383	23·706	21·711	20·207	17·929
17	31·765	27·794	24·062	22·037	20·515	18·198
17·5	32·229	28·200	24·413	22·358	20·815	18·465
18	32·686	28·600	24·760	22·676	21·110	18·726
18·5	33·137	28·995	25·101	22·988	21·391	18·985
19	33·582	29·384	25·438	23·298	21·688	19·239
19·5	34·021	29·768	25·771	23·602	21·991	19·491
20	34·454	30·147	26·091	23·902	22·251	19·739

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS. For natural velocity. 1	For narrow openings of bridges. ·9	For velocity of approach. ·8	For special weirs. ·7	For special orifices. ·6	For broad-crested dams. ·5
	Velocities of Discharge.					
20·5	36·336	32·702	29·068	25·435	21·801	18·168
21	36·776	33·098	29·420	25·743	22·066	18·388
21·5	37·211	33·490	29·768	26·047	22·327	18·605
22	37·641	33·877	30·112	26·348	22·585	18·820
22·5	38·067	34·260	30·453	26·646	22·840	19·033
23	38·487	34·647	30·797	26·948	23·098	19·298
23·5	38·903	35·012	31·122	27·232	23·342	19·451
24	39·315	35·383	31·452	27·520	23·589	19·657
24·5	39·723	35·750	31·778	27·806	23·834	19·861
25	40·126	36·113	32·100	28·088	24·075	20·063
25·5	40·525	36·472	32·420	28·367	24·315	20·262
26	40·921	36·379	32·737	28·644	24·553	20·460
26·5	41·312	37·180	33·049	28·918	24·787	20·656
27	41·700	37·530	33·360	29·190	25·020	20·850
27·5	42·084	37·875	33·667	29·458	25·250	21·042
28	42·465	38·218	33·972	29·725	25·479	21·232
28·5	42·843	38·558	34·275	29·990	25·706	21·421
29	43·216	38·890	34·569	30·248	25·927	21·606
29·5	43·588	39·229	34·870	30·511	26·153	21·794
30	43·956	39·560	35·164	30·779	26·374	21·978
30·5	44·320	39·888	35·456	31·024	26·592	22·160
31	44·682	40·213	35·745	31·277	26·809	22·340
31·5	45·041	40·537	36·032	31·528	27·025	22·520
32	45·397	40·857	36·317	31·778	27·238	22·698
32·5	45·751	41·176	36·601	32·025	27·451	22·875
33	46·101	41·491	36·880	32·270	27·660	23·050
33·5	46·449	41·804	37·159	32·514	27·869	23·224
34	46·794	42·114	37·435	32·755	28·076	23·397
34·5	47·137	42·423	37·709	32·996	28·282	23·568
35	47·478	42·730	37·982	33·234	28·487	23·739

N.B.—For overfalls, reduce the tabular velocity by one-third.

TABLE IX.—*continued.*

Head in feet.	CO-EFFICIENTS.					
	For wide bridge-openings. ·96	For lock sluices. ·84	For special weirs. ·727	For weirs generally. ·666	For orifices generally. ·62	For special weirs. ·55
	Velocities of Discharge.					
20·5	34·882	30·522	26·423	24·199	22·528	19·985
21	35·305	30 892	26·737	24·493	22·701	20·227
21·5	35·723	31·257	27·060	24·783	22 971	20·465
22	36·136	31·619	27·373	25·069	23·337	20·702
22·5	36·544	31·976	27·682	25·353	23·601	20·936
23	36·948	32·329	27·988	25·633	23·868	21·228
23·5	37·347	32·679	28·291	25·910	24·120	21·396
24	37·743	33·025	28·590	26·184	24·375	21·623
24·5	38·134	33.367	28 886	26·455	24·628	21·847
25	38·521	33·706	29·180	23·724	24·878	22 069
25·5	38·904	34·041	29·470	26 990	25·125	22·288
26	39·284	34·373	29·757	27·253	25·371	22·506
26·5	39·660	34·702	30·042	27·514	25·613	22·722
27	40·032	35·028	30·324	27·761	25·854	22 935
27·5	40·401	35·351	30·604	28·028	26 092	23·146
28	40·767	35·671	30·881	28·282	26·328	23·355
28·5	41·129	35·988	31·155	28·533	26·563	23·563
29	41·488	36·302	31·427	28·782	26·891	23 766
29·5	41·844	36 614	31·697	29·029	27·024	23·973
30	42·197	36·923	31·956	29 274	27·253	24·176
30·5	42·548	37·229	32·230	29·517	27·478	24·376
31	42·895	37·533	32·493	29·758	27·703	24·574
31·5	43·240	37·835	32·754	29·997	27·925	24·772
32	43·581	38·134	33·013	30·234	28·146	24·968
32·5	43·920	38·430	33·270	30·470	28·365	25·162
33	44·257	38·725	33·525	30·703	28·582	25·355
33·5	44·591	39·017	33·778	30·935	28·798	25·546
34	44·923	39·307	34·029	31·165	29·012	25·737
34·5	45·252	39.595	34·278	31·393	29·225	25·925
35	45·578	39·881	34·526	31·620	29·436	26·113

***N.B.*—For overfalls, reduce the tabular velocity by one-third.**

EXPLANATORY EXAMPLES TO TABLE IX.

Example 1.

An orifice 6 inches in diameter, has its centre under a head of 5 feet of water; required its discharge.

For a circular orifice using ·62 for a co-efficient, the velocity of discharge is 11·121 feet per second, and the sectional area, according to Part 7, Table XII., being ·1963, the discharge $= \cdot 1963 \times 11 \cdot 121 =$ 2·1836 cubic feet per second.

Example 2.

A rectangular orifice is 8 inches broad and 4 inches deep, and is under a head of 4 feet 3 inches; required its discharge.

Since the breadth is greater than the depth, a special co-efficient is required (*See* Co-efficients in Table XII.).

$$\text{Here } \frac{H}{L} = \frac{4 \cdot 25}{\cdot 66} = 7 \text{ approximately, and } \frac{D}{L} = \frac{\cdot 33}{\cdot 66} = \cdot 5.$$

These require a co-efficient ·612, which must hence be applied to the tabular discharge for natural velocity due to the co-efficient 1·00 $\therefore$ the discharge $= 16 \cdot 544 \times \cdot 22 \times \cdot 612 = 2 \cdot 227$ cubic feet per second.

Example 3.

The fall of water through a bridge, having a sectional area of 500 square feet, is ·05 feet; required the discharge.

Take ·96 as a co-efficient for a wide opening, and we get the discharge $= 1 \cdot 758 \times 500 = 879$ cubic feet per second.

Example 4.

The difference of level between the upper and lower ponds of a canal is 6 feet, and the communicating sluice is 2 feet square; required its discharge.

Using the co-efficient ·84 and height 6, the discharge is $16{\cdot}512 \times 4 = 66{\cdot}048$ cubic feet per second.

The effective head gradually decreasing, the mean discharge due to the height is 33·024 cubic feet per second.

If the lock is 60 long and 20 broad, it will hold 7,200 cubic feet of water, and at the above rate will be filled in 218 seconds, or about three minutes and a half.

Example 5. Required the diameter of a vertical pipe to discharge 2 cubic feet per second from a reservoir under a head of 30 feet.

Using the co-efficient ·84, we obtain from the Table 36·923 as velocity of discharge.

The section will then $= \dfrac{2\cdot}{36{\cdot}923} =$ 05417 square feet,

which will require a diameter of 3¼, or practically, 4 inches.

Example 6. Required the length of a weir to discharge 5696 cubic feet per second, at a depth or head from still water to sill of 4 feet.

With a co-efficient ·666, the tabular velocity of discharge is 10·689, from which one-third has to be deducted to obtain the mean velocity of discharge over a weir.

Hence $V = 10{\cdot}689 - 3{\cdot}563 = 7{\cdot}126$ feet per second,

and the section $= \dfrac{5696}{7{\cdot}126} =$ nearly 800 feet;

hence the length $= \dfrac{\text{section}}{\text{depth}} =$ nearly 200 feet.

Example 7. A river passes over a drowned weir: the upper level of water is 3 feet above the lower level, and is 4 feet above the sill of the weir, which is 100 feet long; required the discharge.

The upper portion may be considered as a simple overfall with a head $H = 3$, and with a co-efficient ·666: the lower portion as an orifice, with the same head, but a co-efficient ·62.

According to the Table the velocity of discharge for the one is $9{\cdot}257 - 3{\cdot}086 = 6{\cdot}171$ feet per second; and that for the other is 8·618 feet per second. Hence the discharge:

$$= 50\,(6{\cdot}171 \times 3 + 8{\cdot}618 \times 1) = 50 \times 27{\cdot}131$$
$$= 1356 \text{ cubic feet per second.}$$

EXAMPLE 8. It is required to raise the upper portion of a river 1·5 feet by means of a drowned weir across it. The river has a discharge of 812 cubic feet per second, and a width of 70 feet; what must be the height of the dam—1st, neglecting velocity of approach; 2nd, taking it at 2·5 feet per second?

1st. Let d = depth of sill of dam below the lower water.

Then V = velocity of upper portion, or true overfall;
= $\frac{2}{3}$ velocity for head 1·5 to a co-efficient ·666;
= 4·364 feet per second (from Table);
and V^1 = velocity of lower portion of orifice;
= velocity for a head 1·5 to a co-efficient ·62;
= 6·109 feet per second (from Table).

Then the total discharge 812, is as in the last Example

$$= 70\left\{V \times 1{\cdot}5 + V^1 \times d\right\} = 70\,(6{\cdot}546 + d \times 6{\cdot}109)$$

hence $d = \dfrac{5{\cdot}054}{6{\cdot}109} = {\cdot}827$ feet.

2nd. Taking into consideration the velocity of approach and modifying the co-efficients (*vide* Table XII.) accordingly.

The head due to velocity of approach 2·5 feet per second, for a co-efficient ·8, is from Table IX. about ·15 feet.

Hence the modified co-efficient for overfall will be

$$m\left\{\left\{1+\frac{h^{\frac{3}{2}}}{H}\right\}-\left\{\frac{h^{\frac{3}{2}}}{H}\right\}\right\} = {\cdot}666\left\{\left\{1+\frac{{\cdot}15^{\frac{3}{2}}}{1.5}\right\}-\left\{\frac{{\cdot}15^{\frac{3}{2}}}{1{\cdot}5}\right\}\right\}$$

$$= {\cdot}666\left\{(1{\cdot})^{\frac{3}{2}}-({\cdot}1)^{\frac{3}{2}}\right\} = {\cdot}745$$

and the modified co-efficient for orifice will be

$$m\sqrt{\left\{1+\frac{{\cdot}15}{1{\cdot}5}\right\}} = m\sqrt{1{\cdot}1} = {\cdot}62 \times 1{\cdot}049 = {\cdot}648$$

Making use of these two co-efficients instead of ·666 and ·62 as in the first portion of the Example, we obtain other values.

$$V = 4{\cdot}894;\ \text{and } V^1 = 6{\cdot}385$$

hence $\dfrac{812}{70} = 11{\cdot}6 = 1{\cdot}5\,V + d\,V^1 = 7{\cdot}341 + d \times 6{\cdot}385$

and $d = \dfrac{4{\cdot}259}{6{\cdot}385} = {\cdot}667$ feet.

BENDS AND OBSTRUCTIONS.

TABLE X.

PART I.—Giving loss of head in feet due to bends in pipes corresponding to certain discharges.—(Weisbach formula.)

PART II.—Giving loss of head due to bends in rivers corresponding to certain velocities.—(Mississippi formula.)

PART III.—Giving approximate rise of water in feet due to obstructions, bridges, weirs, &c.:—(the whole section of water being = 1), and corresponding to certain velocities. — (Dubuat formula.)

TABLE X.—Part 1.

Table giving loss of head due to one bend of 90° for pipes with different discharges. (Weisbach formula.)

Diameter of pipe.		Radius of bend.	Loss of Head of water in feet. ·01	·05	·1	·2	·3	·4	·5	·6	·7	·8	·9	1	2
			Corresponding to discharges in cubic feet per second of												
(1″)	·083	·5	·02	·04	·05	·06	·09	·10	·12	·13	·14	·15	·16	·17	·23
(2″)	·166	·5	·07	·15	·22	·30	·37	·43	·48	·53	·57	·61	·64	·68	·96
(3″)	.25	1·	·15	·34	·49	·69	·84	·97	1·08	1·19	1·28	1·37	1·46	1·53	2·17
(4″)	·33	1·	·26	·59	·84	1·18	1·45	1·66	1·87	2·05	2·21	2·36	2·51	2·64	3·74
(5″)	·416	1·5	·43	·96	1·33	1·93	2·36	2·66	3·05	3·34	3·61	3·86	4·00	4·31	6·09
(6″)	·5	1·5	·61	1·35	1·92	2·71	3·32	3·83	4·29	4·69	5·07	5·42	5·75	6·06	8·55
(7″)	·583	1·5	·81	1·81	2·56	3·62	4·34	5·12	5·73	6·28	6·78	7·25	7·09	8·10	11·46
(8″)	·66	1·5	1·06	2·38	3·34	4·76	5·83	6·69	7·52	8·24	8·90	9·52	10·03	10·64	15·05
(9″)	·75	1·5	1·32	2·94	4·16	5·89	7·21	8·33	9 31	10·20	11·02	11·78	12·49	13·17	18·58
(10″)	·83	1·5	1·57	3·52	4·98	7·04	8·63	9·96	11·13	12·20	13·18	14·00	14·94	15·75	22·27
(11″)	·916	1·5	1·89	4·22	5·96	8·43	10·33	11·92	13·34	14·61	15·78	16·87	17·89	18·86	26·67
(12″)	1·0	1·75	2·27	5·08	7·18	10·16	12·44	14·36	16·06	17·60	19·01	20·32	21·55	22·71	32·12
	1·25	2·	3·4	7·7	10·9	15·4	18·9	21·8	24·4	26·7	28·9	30·9	32·7	34·5	48·1
	1·5	2·5	5·0	11·3	15·9	22·6	27·6	31·9	35·7	39·1	42·2	45·1	47·9	50·4	71·4
	1·75	3·	6·9	15·5	21·9	30·9	37·9	43·7	48·9	53·5	57·8	61·8	65·6	69·1	97·7
	2·	4·	9·4	20 9	29·6	41·9	51·3	59·2	66·2	72·5	78·3	83·7	88·8	93·6	132 4
	2·5	5·	14·6	32·7	46·2	65·4	80·1	92·4	103·4	113 3	122·4	130·9	138·8	146·3	206·9
	3·0	6·	21·1	47·1	66·6	94·2	115·4	133·2	148·9	163·2	176·3	188·4	199·9	210·7	297·9
	3·5	7·	28·7	64·1	90·7	128·2	157·1	181·3	202·8	222·1	230·9	203·7	272·0	286·7	405·5
	4·	8·	37·3	83·9	118·7	167·9	205·6	237·4	265·4	290·8	314·1	335·8	356·1	375·4	530·9
	5·	10·	58·5	130·9	185·1	261·7	320·5	370·1	413·8	453·3	489·6	523·4	555·2	585·2	827·6

TABLE X.—Parts 2 and 3.

Part 2.—Bends of Rivers. (Mississippi formula.)

For velocities in feet per second.	For deflections of 10° $\frac{n}{3}$	20° $\frac{n}{2}$	30° n	60° 2 n	90° 3 n	180° 6 n
	Loss of head in feet.					
1	·0006	·0009	·0019	·0037	·0096	·0112
2	·0025	·0037	·0075	·0149	·0234	·0448
3	·0056	·0084	·0168	·0336	·0503	·1007
4	·0099	·0149	·0298	·0597	·0895	·1790
5	·0155	·0233	·0466	·0933	·1399	·2798
6	·0224	·0335	·0671	·1343	·2014	·4028
7	·0305	·0457	·0914	·1827	·2741	·5483
8	·0398	·0596	·1194	·2387	·3581	·7162
9	·0503	·0755	·1511	·3021	·4532	·9064
10	0622	·0933	·1865	·3730	·5592	1·1190

Part 3.—Obstructions (Dubuat formula) when the hydraulic slope is less than ·001.

For velocities in feet per second.	For percentages of obstruction to whole channel section. ·1	·2	·3	·4	·5	·6
	Rise resulting in feet.					
1	·004	·009	·018	·031	·051	·089
2	·015	·034	·070	·120	·203	·355
3	·035	·085	·158	·270	·456	·798
4	·062	·150	·282	·480	·811	1·419
5	·097	·236	·439	·752	1·267	2·218
6	·140	·341	·634	1·080	1·824	3·193
7	·191	·463	·862	1·470	2·484	4·346
8	·249	·602	1·126	1·920	3·245	5·677
9	·315	·766	1·426	2 430	4·107	7·185
10	·389	·956	1·758	3·008	5·070	8·872

Explanatory Examples to Table X.

Example 1. A series of pipes have to discharge 5 gallons per second; there are 7 bends in the portion that consists of 5-inch pipe, 4 in that of 6-inch pipe, and 8 in that of 7-inch pipe; what is the total loss of head on account of these bends?

From Table XI. 5 gallons per second = ·8 cubic feet per second, and taking the heads separately from Table X.

				feet.
7 bends in 5 inch give	7 × ·045	=		·315
4 „ „ 6 „ „	4 × ·030	=		·120
8 „ „ 7 „ „	8 × ·010	=		·080
	Total loss of head	=		·515

The head on the pipes must therefore not only be sufficient to force ·8 cubic feet per second through the pipes under ordinary conditions, but must also be increased by ·515 on account of bends.

Example 2. A river has one bend of 20°, two of 30°, and one of 90°, what is the total loss of head expended in overcoming these bends, when the velocity is 5 feet per second?

From Part 2, Table XII.

1 bend of 20° gives	1 × ·0233	=	·0233
2 „ „ 30° „	2 × ·0466	=	·0932
1 „ „ 90° „	1 × ·1399	=	·1399
Total head expended		=	·2564 feet.

Example 3. A river having a hydraulic slope less than ·001 has its section obstructed by the piers and abutments of a bridge to the extent of one-fifth, the normal velocity being 3·5 feet per second, what is the rise caused by the bridge?

By Part 3, Table XI., the rise will be ·12 feet.

N.B.—For rivers having steeper gradients, apply a correction according to the formula given in the text.

TABLE XI.

TABLE OF EQUIVALENTS.

PART 1.—Equivalent supply from total quantities.

2.—Equivalent discharges.

3.—Equivalent velocities.

4.—Equal discharging channels.

5.—Conversion tables for English measures.

6.—Conversion tables for metrical measures.

TABLE XI.—Part I.

Equivalent Supply.

Continuous supply in cubic feet per second into total quantities and vice versâ.

Total quantity in cubic feet.	Continuous supply in cubic feet per second.					
	For 2 months.	For 3 months.	For 6 months.	For 8 months.	For 9 months.	For 12 months.
315 360	·06	·04	·02	·015	·013	·01
630 720	·12	·08	·04	·030	·026	·02
946 080	·18	·12	·06	·045	·040	·03
1 261 440	·24	·16	·08	·060	·053	·04
1 576 800	·30	·20	·10	·075	·066	·05
1 892 160	·36	·24	·12	·090	·080	·06
2 207 520	·42	·28	·14	·105	·093	·07
2 522 880	·48	·32	·16	·120	·106	·08
2 838 240	·54	·36	·18	·135	·120	·09
1 million	·1903	·1268	·0634	·0476	·0423	·031710
2 millions	·3805	·2537	·1268	·0851	·0846	·063420
3 „	·5708	·3805	·1903	·1427	·1268	·095129
4 „	·7610	·5074	·2537	·1902	·1691	·126839
5 „	·9513	·6342	·3171	·2378	·2114	·158549
6 „	1·1416	·7610	·3805	·2854	·2537	·190259
7 „	1·3318	·8879	·4439	·3119	·2960	·221969
8 „	1·5221	1·0147	·5074	·3405	·3382	·253678
9 „	1·7123	1·1416	·5708	·4280	·3805	·285388
10 „	1·9026	1·2684	·6342	·4756	·4228	·317098

TABLE XI.—Part 1—*continued.*

Equivalent Supply.

Continuous supply in cubic feet per second throughout a month that is equivalent to a certain number of waterings in a month.

Amounts given at each watering to one acre.	At 30 waterings per month.	At 15 waterings per month.	At 10 waterings per month.	At 4 waterings per month.	At 2 waterings per month.	At 1 watering per month.
Cubic feet.	Monthly supply in cubic feet per second.					
10000	·1157	·0579	·0386	·0154	·0077	·0039
9000	·1041	·0520	·0347	·0139	·0069	·0035
8000	·0926	·0463	·0309	·0123	·0062	·0031
7000	·0810	·0405	·0271	·0108	·0054	·0027
6000	·0694	·0347	·0231	·0092	·0046	·0023
5000	·0579	·0289	·0193	·0077	·0039	·0019
4000	·0463	·0231	·0154	·0062	·0031	·0015
3000	·0347	·0173	·0116	·0046	·0023	·0011
2000	·0231	·0116	·0077	·0031	·0015	·0008
1000	·0116	·0058	·0039	·0015	·0008	·0004
8640	·1	·050	·033	·013	·0066	·0033
7776	·09	·045	·030	·012	·0060	·0030
6912	·08	·040	·027	·011	·0054	·0027
6048	·07	·035	023	·009	0046	·0023
5184	·06	·030	·020	·008	·0040	·0020
4320	·05	·025	·016	·006	·0032	·0016
3456	·04	·020	·013	·005	·0026	·0013
2592	·03	·015	·010	·004	·0020	.0010
1728	·02	·010	·007	·003	·0014	·0007
864	·01	·005	·003	·001	·0007	·0003

N.B.—In this table a month of 30 days is assumed.

TABLE XI.—Part 2.

Equivalent Discharges.

Cubic feet per second, per minute, and per day, into Gallons per second, per minute, and per day.

Per second.		Per minute.		Per day of 24 hours.	
Cubic feet.	Gallons.	Cubic feet.	Gallons.	Cubic feet.	Gallons.
·01	·06	·6	3·74	864	5384
·02	·12	1·2	7·47	1728	10768
·03	·19	1·8	11·21	2592	16152
·04	·25	2·4	14·95	3436	21536
·05	·31	3·	18·69	4320	26920
·06	·37	3·6	22·43	5184	32304
·07	·44	4·2	26·17	6048	37688
·09	·5	4·8	29·90	6912	43072
·09	·56	5·4	33·64	7776	48456
·1	·62	6·	37·39	8640	53844
·16	1·04	10·	62·32	14400	89741
·33	2·08	20·	124·64	28800	179842
·5	3·12	30·	186·96	43200	269223
·66	4·16	40·	249·28	47600	358964
·83	5·20	50·	311·60	72000	448704
1·	6·23	60·	373·92	86400	538446
1·16	7·27	70·	436·24	100800	628187
1·33	8·31	80·	498·56	115200	717928
1·5	9·35	90·	560·88	129600	807669
1·66	10·39	100·	623·20	144000	897408
1·15	7·21	69·4	432.7	100000	623200
1·93	14·42	115·7	865·4	200000	1246400
3·47	21·63	208·3	1298·1	300000	1869600
4·63	28·84	277·7	1730·8	400000	2492800
5·78	36·05	346·8	2163·5	500000	3116000
6·94	43·26	416·6	2596·2	600000	3739200
8·10	50·47	486·	3028·9	700000	4362400
9·26	57·68	555·5	3461·6	800000	4985600
10·41	64·89	624·9	3894 3	900000	5608800
11·57	72·10	694·4	4327·5	1 million.	6232000

TABLE XI.—Part 2—*continued.*

Equivalent Discharges.

Gallons per second, per minute, and per day, into Cubic feet per second, per minute, and per day.

Per second.		Per minute.		Per day of 24 hours.	
Gallons.	Cubic ft.	Gallons.	Cubic ft.	Gallons.	Cubic feet.
·1	·01	6	·96	8640	1385
·2	·03	12	1·92	17280	2772
·3	·05	18	2·88	25920	4158
·4	·06	24	3·84	34360	5543
·5	·08	30	4·80	43200	6929
·6	·09	36	5·76	51840	8315
·7	·11	42	6·72	60480	9701
·8	·13	48	7·68	69120	11087
·9	·14	54	8·64	77760	12473
1·	·16	60	9·62	86400	13858
·166	·03	10	1·60	14400	2310
·333	·05	20	3·21	28800	4619
·5	·08	30	4·81	43200	6929
·666	·11	40	6·42	57600	9239
·833	·14	50	8·02	72000	11549
1·	·16	60	9·62	86400	13858
1·166	·19	70	11·23	100800	16168
1·333	·21	80	12·83	115200	18478
1·5	·24	90	14·44	129600	20788
1·666	·26	100	16·04	144000	23097
1·15	·186	69·4	111·4	100000	16040
1·93	·371	115·7	222·8	200000	32079
3·47	·557	208·3	334·2	300000	48119
4·63	·742	277·7	445·6	400000	64159
5·78	·928	346·8	556·9	500000	80199
6·94	1·114	416·6	667·3	600000	96239
8·10	1·299	486·	779·7	700000	112278
9·26	1·485	555·5	891·1	800000	128318
10·41	1·670	624·9	1002·5	900000	144358
11·57	1·856	694·4	1113·9	1 million	160398

TABLE XI.—Part 3.

Equivalent Velocities and Heads for Natural Velocities.

Feet per second.	Feet per minute.	Miles per hour.	Corrdg. head of water.	Feet per second.	Feet per minute.	Miles per hour.	Corrdg. head of water.
1	60	·6818	·016	3·5	210	2·3866	·191
1·1	66	·7500	·019	3·6	216		·204
1·2	72	·8181	·023	3·7	222		·214
1·3	78	·8863	·026	3·8	228		·224
1·4	84	·9545	·031	3·9	234		·238
1·5	90	1·0238	·035	4	240	2·7264	·250
1·6	96		·040	4·1	246		·263
1·7	102		·045	4·2	252		·275
1·8	108		·051	4·3	258		·289
1·9	114		·056	4·4	264		·303
2	120	1·364	·063	4·5	270	3·0672	·317
2·1	126		·069	4·6	276		·331
2·2	132		·076	4·7	282		·345
2·3	138		·083	4·8	288		·360
2·4	144		·092	4·9	294		·375
2·5	150	1·706	·098	5	300	3·4091	·391
2·6	156		·103	5·1	306		·406
2·7	162		·115	5·2	312		·422
2·8	168		·124	5·3	318		·441
2·9	174		·131	5·4	324		·459
3	180	2·048	·141	5·5	330	3·7500	·473
3·1	186		·151	5·6	336		·495
3·2	192		·160	5·7	342		·508
3·3	198		·170	5·8	348		·523
3·4	204		·180	5·9	354		·544
3·5	210	2·3866	·191	6·	360	4·089	·562

TABLE XI.—PART 3—*continued.*

Feet per second.	Feet per minute.	Miles per hour.	Corrsdg. head of water.
6·1	366		·581
6·2	372		·603
6·3	378		·620
6·4	384		·640
6·5	390	4·2045	·660
6·6	396		·680
6·7	402		·701
6·8	408		·720
6·9	414		·744
7	420	4·771	·766
7·1	426		·787
7·2	432		·816
7·3	438		·832
7·4	444		·856
7·5	450	5·1136	·879
7·9	456		·896
7·7	462		·926
7·8	468		·952
7·9	474		·975
8	480	5·453	1
8·2	492		1·052
8·4	504		1·100
8·6	516		1·146
8·8	528		1·212
9	540	6·1363	1·265

Feet per second.	Feet per minute.	Miles per hour.	Corrsdg. head of water.
9·2	552		1·324
9·4	564		1·380
9·6	576		1·460
9·8	588		1·500
10	600	6·818	1·564
10·2	612		1·624
10·4	624		1·644
10·6	636		1·764
10·8	648		1·836
11	660	7·500	1·892
11·2	672		1·930
11·4	684		2·032
11·6	696		2·092
11·8	708		2·176
12	720	8·1727	2·252
13	780	8·8636	2·6531
14	840	9·5454	3·0625
15	900	10·2272	3·5156
16	960	10·9090	4
17	1020	11·5909	4·5156
18	1080	12·2727	5·0625
19	1140	12·9545	5·6406
20	1200	13·6363	6·25
30	1800	20·4545	14·0625
40	2400	27·2727	25
50	3000	34·0909	39·062

TABLE XI.—Part 4.

Table of equal-discharging Trapezoidal Channel-Sections (from Stoddard).
Giving depths corresponding to mean widths for any unit;—Slopes 0 to 1 to 2 to 1.

Mean Widths.									
70	60	50	40	35	30	25	20	15	10
Relative Depths.									
·25	·27	·30	·35	·40	·45	·52	·58	·71	·98
·50	·55	·62	·73	·80	·89	1·02	1·19	1·48	2·04
·75	·82	·94	1·10	1·20	1·35	1·56	1·82	2·28	3·22
1	1·10	1·26	1·48	1·62	1·81	2·10	2·46	3·11	4·50
1·25	1·39	1·58	1·86	2·04	2·28	2·65	3·12	3·98	5·89
1·50	1·67	1·90	2·24	2·46	2·75	3·20	3·80	4·88	7·31
1·75	1·95	2·22	2·62	2·88	3·23	3·75	4·50	5·80	8·86
2	2·23	2·54	3	3·31	3·72	4·32	5·22	6·78	10·50
2·25	2·51	2·86	3·38	3·73	4·21	4·91	5·95	7·81	12·25
2·50	2·79	3·18	3·76	4·16	4·70	5·50	6·68	8·84	14
2·75	3·07	3·51	4·15	4·60	5·21	6·09	7·45	9·93	15·84
3	3·35	3·84	4·54	5·04	5·72	6·69	8·22	11·03	17·68
3·50	3·91	4·5	5·33	5·95	6·75	7·93	9·82	13·32	21·68
4·	4·48	5·14	6·13	6·85	7·81	9·21	11·48	15·75	26
4·50	5·05	5·79	6·95	7·75	8·90	10·50	13·19	18·22	30·36
5	5·62	6·45	7·75	8·66	10	11·79	14·96	20·80	35
5·50	6·18	7·12	8·57	9·62	11·10	13·24	16·77	23·47	39·81
6	6·75	7·80	9·40	10·60	12·22	14·65	18·65	26·65	44·86

TABLE XI.—Part 4—*continued.*

Discharge in cubic feet per second.		Data of channels of equal discharge (friction-co-efficient f = ·03) for canals in moderately good order in earth with side slopes of 1½ to 1. S per 1000 is the fall in 1000; D the depth; B the bottom breadth; V mean velocity.											
10	S per 1000	0·1	0·5	1·0	1·5	2·0	2·5	3·0	·1	·2	·3	·4	·05
	D	1·31	1·31	1·31	1·31	1·31	1·31	1·31	1·97	1·97	1·97	1·97	2·62
	B	14·60	5·91	3·95	2·83	2·53	2·16	1·87	6·41	4·03	2·95	2·23	4·92
	V	·464	·965	1·282	1·502	1·680	1·827	1·955	·541	·728	·855	·945	·397
100	S per 1000	1·4	3·	·5	1·0	1·6	2·4	3·0	·1	·5	1·0	·1	·05
	D	1·97	1·97	2·62	2·62	2·62	2·62	2·62	3·28	3·28	3·28	3·94	4·59
	B	17·45	11·78	17·85	12·20	9·35	7·32	6·33	27·33	11·32	7·28	19·46	20·57
	V	2·490	8·458	1·748	2·359	2·864	3·373	3·704	·942	1·875	2·490	1·012	·794
1000	S per 1000	1·2	0·5	0·2	0·3	0·4	0·5	0·7	1·0	·1	·2	·1	·05
	D	4·59	5·25	5·91	5·91	5·91	5·91	5·91	5·91	6·56	6·56	7·22	7·87
	B	44·19	54·79	70·77	57·81	49·74	45·97	36·98	30·42	82·35	58·43	69·52	82·32
	V	4·255	3·038	2·129	2·543	2·890	3·195	3·691	4·311	1·653	2·231	1·722	1·358
10000	S per 1000	1·8	1·0	·7	·4	·2	·1	·02	·03	·05	·02	·03	·02
	D	8·53	9·19	9·84	11·48	13·12	14·76	16·40	16·40	16·40	18·04	18·04	19·69
	B	129·46	158·94	172·15	165·91	184·71	206·83	325·20	279·56	229·69	274·12	235·96	233·96
	V	8·238	6·489	5·584	4·751	3·730	2·956	1·742	2·001	2·392	1·837	2·106	1·929

TABLE XI.—Part 5.

Conversion of English Measures.

Linear—

Inches and eighths in feet.

Inches.	0	1/8	2/8	3/8	4/8	5/8	6/8	7/8
0	...	·010	·021	·031	·042	·052	·063	·073
1	·083	·094	·104	·114	·125	·135	·146	·156
2	·166	·177	·187	·198	·208	·219	·229	·239
3	·250	·260	·271	·281	·291	·302	·312	·323
4	·333	·344	·354	·364	·375	·385	·396	·406
5	·416	·427	·437	·448	·458	·469	·479	·489
6	·5	·510	·520	·531	·541	·552	·562	·573
7	·583	·594	·604	·614	·625	·635	·646	·656
8	·666	·677	·687	·697	·708	·718	·729	·739
9	·75	·760	·771	·781	·791	·802	·812	·823
10	·833	·844	·854	·864	·875	·885	·895	·906
11	·916	·927	·937	·948	·958	·969	·979	·989

Superficial—

Acres into square feet and vice versâ.

Square feet.	Acres.	Acres.	Square feet.
1000	·022957	1	43560
2000	·045914	2	87120
3000	·068870	3	130680
4000	·091827	4	174240
5000	·114784	5	217800
6000	·137742	6	261360
7000	·160697	7	304920
8000	·183656	8	348480
9000	·206613	9	392040
10000	·229568	10	435600

TABLE XI.—PART 5—*continued.*

Equivalent English measures.

Square miles into acres and vice versâ.

Acres.	Square miles.	Square miles.	Acres
1	·001563	1	640
2	·003135	2	1280
3	·004688	3	1920
4	·006270	4	2560
5	·007813	5	3200
6	·009375	6	3840
7	·010938	7	4480
8	·012540	8	5120
9	·014063	9	5760
10	·015625	10	6400

Capacity—

Cubic feet into gallons and vice versâ.

Cubic feet.	Gallons.	Gallons.	Cubic feet.
1	6·232	1	·1605
2	12·464	2	·3209
3	18·696	3	·4813
4	24·928	4	·6418
5	31·160	5	·8023
6	37·392	6	·9627
7	43·624	7	1·1232
8	49·856	8	1·2836
9	56·088	9	1·4441
10	62·320	10	1·6046

TABLE XI.—PART 6.

Conversion into Metrical measures.

Linear Measures.

Units.	Feet into metres.	Inches into centimetres.	Miles into kilometres.	Units.	Metres into feet.	Centimetres into inches.	Kilometres into miles.
1	·30479	2·5400	1·6093	1	3·2809	·39371	·6214
2	·60959	5·0799	3·2186	2	6·5618	·78742	1·2428
3	·91438	7·6199	4·8279	3	9·8427	1·18112	1·8641
4	1·21918	10·1598	6·4372	4	13·1236	1·57483	2·4855
5	1·52397	12·6998	8·0464	5	16·4045	1·96854	3·1069
6	1·82877	15·2397	9·6557	6	19·6854	2·36225	3·7283
7	2 13356	17·7797	11·2650	7	22·9663	2·75596	4·3496
8	2·43836	20·3196	12·8743	8	26·2472	3·14966	4·9710
9	2·74315	22·8596	14·4836	9	29·5281	3·54337	5·5924
10	3·04794	25·3995	16 0922	10	32·8090	3·93708	6·2138

Superficial Measures.

Units.	Sq. feet into sq. metres.	Acres into hectares.	Sq. miles into sq. kilometres.	Units.	Sq. metres into sq. feet.	Hectares into acres.	Sq. kilometres into sq. miles.
1	·0929	·4047	2·5898	1	10·7643	2·4712	·3861
2	·1858	·8093	5·1797	2	21·5286	4·9424	·7722
3	·2787	1·2140	7·7695	3	32·2929	7·4136	1·1583
4	3716	1·6186	10·3594	4	43·0572	9·8848	1·5444
5	·4645	2 0233	12 9492	5	53·8215	12·3560	1·9305
6	·5574	2·4280	15·5391	6	64·5858	14·8272	2·3167
7	·6503	2·8329	18·1289	7	75·3501	17·2984	2·7028
8	·7432	3·2373	20·7188	8	86·1144	19·7696	3·0889
9	·8361	3·6420	23·3086	9	96·8787	22·2408	3·4750
10	·9290	4·0466	25·8985	10	107·6430	24·7120	3·8611

TABLE XI.—Part 6—*continued.*

Measures of Capacity.

Units.	Cub. feet into cub. metres.	Gallons into litres.	Bushels into hectolitres.	Units.	Cub. metres into cub. feet.	Litres into gallons.	Hectolitres into bushels.
1	·028315	4·5435	·3635	1	35·317	·22010	2·751
2	·056630	9·0879	·7270	2	70·633	·44019	5·502
3	·084945	13·6304	1·0904	3	105·950	·66029	8·253
4	·113260	18·1738	1·4539	4	141·266	·88039	11·004
5	·141575	22·7173	1·8174	5	176·583	1·10048	13·756
6	·169890	27·2607	2·1808	6	211·900	1·32058	16·506
7	·198205	31·8042	2·5445	7	247·216	1·54068	19·257
8	·226520	36·3476	2·9078	8	282·533	1·76077	22·008
9	·254835	40 8911	3·2712	9	317·849	1·98087	24 759
10	·283150	45·4345	3·6348	10	353·166	2·20097	27·512

Weights.

Units.	Grains troy into grammes.	Pounds into kilogrammes.	Tons into quintals.	Units.	Grammes into grains troy.	Kilogrammes into pounds.	Quintals into tons.
1	·0648	·4535	10·159	1	15·434	2·2048	·0984
2	·1295	·9070	20·318	2	30·868	4·4096	·1969
3	·1943	1·3605	30·477	3	46·302	6·6144	·2953
4	·2591	1·8140	40·636	4	61·736	8·8192	·3938
5	·3239	2·2675	50·797	5	77·170	11·0240	·4922
6	·3886	2·7210	60·954	6	92·604	13·2288	·5907
7	·4534	3·1745	71·113	7	108·038	15·4336	·6891
8	·5182	3·6280	81·272	8	123·472	17·6384	·7876
9	·5829	4·0815	91·436	9	138·906	19·8432	·8856
10	·6478	4·5350	101·595	10	154·340	22·0480	·9844

TABLE XI.—Part 6—*continued.*

Measures of Water Supply.

A watering in cubic feet per acre of		A watering in cubic metres per hectare of	A watering in cubic metres per hectare of		A watering in cubic feet per acre of
10000	=	700	1000	=	14284
9000	=	630	900	=	12852
8000	=	560	800	=	11424
7000	=	490	700	=	9996
6000	=	420	600	=	8568
5000	=	350	500	=	7142
4000	=	280	400	=	5712
3000	=	210	300	=	4284
2000	=	140	200	=	2856
1000	=	70	100	=	1428

A supply in litres per second per hectare of		A supply in cubic feet per second per acre of	A supply in cubic feet per second per acre of		A supply in litres per second per hectare of
2·00	=	·02856	·03	=	2·10018
1·50	=	·02142	·02	=	1·40012
1·00	=	·01428	·01	=	·70006
·75	=	·01021	·0075	=	·52004
·5	=	·00714	·0050	=	·35003
·25	=	·00357	·0025	=	·17501

1 litre per second per hectare = a duty of 70 acres per cubic foot per second.

·01 cubic feet per second per acre = a duty of 100 acres per cubic foot per second.

A hectare is equal to 10 000 square metres.

A litre is equal to $\frac{1}{1000}$ of a cubic metre.

TABLE XI.—Part 6—*continued*

Measures of Pressure.

Lbs. per square inch into kilogrammes per square centimetre.		Kilogrammes per square centimetre into lbs. per square inch.	
1	·0703	1	14·237
2	·1406	2	28·475
3	·2109	3	42·713
4	·2812	4	56·950
5	·3515	5	71·187
6	·4218	6	85·426
7	·4921	7	99·663
8	·5624	8	113·901
9	·6327	9	128·138
10	·7023	10	142·375

Measures of Heat.

Centigrade.	Fahrenheit.	Centigrade.	Fahrenheit.	Centigrade.	Fahrenheit.	Centigrade.	Fahrenheit.
0°	32°	20°	68°	25°	77°	30°	86°
2·5	36·5	20·5	68·9	25·5	77·9	30·5	86·9
5	41	21	69·8	26	78·8	31	87·8
7·5	45·5	21·5	70·7	26·5	79·7	31·5	88·7
10	50	22	71·6	27	80·6	32	89·6
12·5	54·5	22·5	72·5	27·5	81·5	32·5	90·5
15	59	23	73·4	28	82·4	33	91·4
17·5	63·5	23·5	74·3	28·5	83·3	33·5	92·3
20	68	24	75·2	29	84·2	34	93·2
		24·5	76·1	29·5	85·1	34·5	94·1
35°	95°	40°	104°	45°	113°	50°	122°
35·5	95·9	40·5	104·9	45·5	113·9	55	131
36	96·8	41	105·8	46	114·8	60	140
36·5	97·7	41·5	106·7	46·5	115·7	65	149
37	98·6	42	107·6	47	116·6	70	158
37·5	99·5	42·5	108·5	47·5	117·5	75	167
38	100·4	43	109·4	48	118·4	80	176
38·5	101·3	43·5	110·3	48·5	119·3	85	185
39	102·2	44	111·2	49	120·2	90	194
39·5	103·1	44·5	112·1	49·5	121·1	100	212°

TABLE XII.

TABLE XII.—Part 1.

Co-efficients of fluid friction, being the values of f in the formula given in the text.

(D'Arcy, Bazin, Ganguillet, and Kutter.)

(From the "Cultur-Ingénieur," 1870.)

General values.

·009—Well planed plank.
·010—Very smooth surfaces, plasters in cement; assumed to be applicable also to enamelled and glazed pipes.
·011—Plaster in cement, with one-third sand.
·012—Unplaned plank.
·013—Brickwork and cut stone; assumed to apply also to metal and earthenware pipes under ordinary conditions, but not new.
·017—Rubble masonry.
·020—Canals with bed and banks of very firm gravel.
·025—Rivers and Canals in Earth, in perfect order and regimen, and perfectly free from stones and weeds.
·030—Rivers and Canals in Earth, in moderately good order and regimen, having stones and weeds occasionally.
·035—Rivers and Canals in Earth, in bad order and regimen, having stones and weeds in great quantity.

Local values.

·019—The Marseilles Canal.
·022—Rigoles de Grosbois.
·023—Tauber Alpbachschale, Rhine.
·024—Linth canal. Hübengraben. Hill-streams.
·025—Jard canal. Seine. Neva.
·026—Seine. Haine. Rhine. Speierbach.
·027—Mississippi. Rhine.
·028—Saone, Salzach.
·029—Danube in Hungary.
·030—Rigoles de Chazilly.
·031—Limat, Zurich.
·033—Maras.
·035—Simme.

TABLE XII.—Part 2.

Co-efficients of flood discharge from catchment areas.

For the formula in Table IV., Part 1, also given in the text.

$$Q = n \times 100 \, (N)^{\frac{3}{4}}$$

The value of this co-efficient (n) can be determined and made use of within local limits only, as it depends on the average maximum local downpour, the evaporation, the quality, inclination, and disposition of the surface of the area under consideration; it has hitherto been determined for very few districts, and not sufficiently satisfactorily for some of those. In some cases, unfortunately, doubtful flood marks have been used to obtain the flood gradient, and the velocities calculated according to very varied formulæ; in others, the obstructions caused by bridges and embankments have vitiated all the bases of calculation of discharge.

	Values of n
For very large Indian rivers near their mouths ...	·03 to 2
For Oudh generally	1 to 2
The Madras Presidency, the whole Cavery; The Godavery, Kistna, Tumbaddra, Pennair, Vigay } about	2·
The Chittaur, Palaur, Manjilanthi, Varhazanthi below ...	5·
For the Kanhan River, Central Provinces, according to the highest flood yet known, less than	5·
For Bengal and Bahar, rainfall 2 to 4 feet—Col. Dickens gives a co-efficient of	8·25
The Upper Cavery, Tambrapurni, Gadanamatti ...	12, 16, and 22·
For some rivers in Berar and the Central Provinces, according to calculated velocities only	16· to 24·

Some further data for Indian rivers will be found in the Statistics.

TABLE XII.—Part 3.

Co-efficients of velocity of discharge for surfaces with a frictional co-efficient $f = \cdot01$, suitable to very smooth plastered channels in cement (Kutter), and to enamelled and glazed pipes.

For values of S.	For values of R in feet.											
	·1	·2	·3	·4	·5	·6	·7	·8	·9	1·	1·25	1·50
·0001	·72	·92	1·03	1·13	1·22	1·26	1·29	1·35	1·38	1·41	1·48	1·54
·0002	·77	·98	1·08	1·17	1·27	1·31	1·35	1·38	1·42	1·44	1·51	1·56
·0003	·81	1·00	1·11	1·19	1·29	1·32	1·36	1·40	1·43	1·44	1·52	1·57
·0004	·82	1·01	1·12	1·21	1·29	1·33	1·37	1·41	1·44	1·46	1·52	1·57
·0005	·82	1·02	1·12	1·22	1·30	1·34	1·38	1·41	1·44	1·47	1·53	1·58
·0008	·83	1·03	1·13	1·22	1·31	1·35	1·38	1·42	1·45	1·48	1·53	1·58
·001 and steeper falls.	·84	1·04	1·14	1·23	1·31	1·35	1·38	1·43	1·46	1·49	1·54	1·58

Applying the above to glazed and enamelled pipes, having gradients steeper than ·001, and neglecting very small pipes, for which experimental data are wanting, the co-efficients will be—

For diameters of	5 inches and under,	6″,	7″,	8″,	9″,	10″,	12″
Co-efficients	·84	·87,	·91,	·95,	·99,	1·04,	1·09.

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces with a frictional co-efficient $f = \cdot 01$, suitable to very smooth plastered channels in cement, and to enamelled and glazed pipes.

For values of S.	For values of R in feet.											
	1·75	2·0	2·5	3·0	3·5	4·0	4·5	5	6	8	10	12
·0001	1·58	1·62	1·68	1·74	1·78	1·82	1·85	1·87	1·91	1·98	2·02	2·06
·0002	1·60	1·63	1·70	1·74	1·77	1·81	1·83	1·85	1·89	1·94	1·98	2·01
·0003	1·61	1·64	1·70	1·74	1·77	1·80	1·83	1·85	1·88	1·93	1·97	1·99
·0004	1·61	1·65	1·70	1·74	1·77	1·80	1·83	1·85	1·88	1·93	1·97	1·99
·0005	1·62	1·65	1·70	1·74	1·77	1·80	1·82	1·84	1·87	1·92	1·96	1·98
·0008	1·63	1·65	1·70	1·74	1·77	1·80	1·82	1·84	1·87	1·92	1·95	1·98
·001 and steeper falls.	1·63	1·65	1·70	1·74	1·77	1·80	1·82	1·84	1·86	1·91	1·95	1·97

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 013$, suitable to channels or culverts in brickwork or cut stone (Kutter), and to metal pipes when not new.

For values of S.	For values of R in feet. ·1	·2	·3	·4	·5	·6	·7	·8	·9	1·0	1·25	1·50
·0001	·50	·66	·74	·84	·90	·94	·96	1·00	1·04	1·05	1·12	1·17
·0002	·55	·71	·79	·87	·94	·97	1·00	1·04	1·07	1·09	1·15	1·19
·0003	·58	·72	·81	·89	·96	·99	1·01	1·05	1·08	1·10	1·15	1·19
·0004	·59	·73	·82	·89	·96	·99	1·02	1·06	1·08	1·10	1·15	1·20
·0005	·60	·74	·83	·90	·97	1·00	1·03	1·07	1·09	1·10	1·16	1·20
·0008	·60	·75	·83	·91	·97	1·01	1·03	1·07	1·09	1·11	1·17	1·20
·001 and steeper falls.	·61	·75	·83	·92	·98	1·01	1·04	1·07	1·10	1·12	1·17	1·21

Applying the above to ordinary metal pipes, having gradients steeper than ·001, and neglecting very small pipes for which experimental data are wanting, the co-efficients will be—

For diameters of	5 inches and under,	6″,	7″,	8″,	9″,	10″,	12
Co-efficients	·61	·63,	·66,	·69,	·72,	·75,	·79.

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 013$—*continued.*

(Kutter.)

For values of S.	For values of R in feet.											
	1·75	2·	2·5	3·	3·5	4·	4·5	5·	6·	8·	10	12
·0001	1·22	1·24	1·30	1·35	1·38	1·41	1·44	1·46	1·50	1·57	1·61	1·64
·0002	1·23	1·26	1·30	1·35	1·38	1·41	1·43	1·45	1·48	1·53	1·57	1·60
·0003	1·23	1·27	1·31	1·35	1·38	1·40	1·43	1·44	1·47	1·52	1·56	1·57
·0004	1·23	1·27	1·31	1·35	1·38	1·40	1·42	1·44	1·47	1·52	1·55	1·57
·0005	1·24	1·27	1·31	1·35	1·38	1·40	1·42	1·43	1·47	1·51	1·55	1·57
·0008	1·24	1·28	1·31	1·35	1·38	1·39	1·42	1·43	1·47	1·51	1·54	1·56
·001 and steeper falls.	1·24	1·28	1·31	1·35	1·38	1·39	1·41	1·43	1·46	1·50	1·53	1·55

TABLE XII.—PART 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 017$, suitable to channels or aqueducts in rubble. (Kutter.)

For values of S.	For values of R in feet.												
	·1	·5	·75	1	2	3	4	5	6	7	8	9	10
.0001	·35	·65	·72	·78	·93	1·03	1·09	1·13	1·17	1·20	1·22	1·25	1·27
·0002	·38	·67	·75	·80	·95	1·03	1·08	1·12	1·16	1·18	1·20	1·22	1·23
·0003	·39	·68	·76	·81	·95	1·03	1·08	1·12	1·15	1·17	1·19	1·21	1·22
·0004	·40	·69	·76	·82	·95	1·03	1·08	1·11	1·15	1·17	1·19	1·20	1·22
·0005	·40	·70	·77	·82	·95	1·03	1·07	1·11	1·14	1·17	1·18	1 20	1·21
·0008	·41	·71	·77	·83	·95	1·03	1·07	1·11	1·14	1·17	1·18	1·19	1·21
·001 and steeper falls.	·41	·71	·78	·83	·96	1·03	1·07	1·11	1·13	1·16	1·17	1·19	1·21

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = {\cdot}025$, suitable to canals and rivers with earthen beds in perfect order and regimen, and perfectly free from stones and weeds. (Kutter.)

For values of S.	For values of R in feet.												
	·5	·75	1	1·5	2	2·5	3	3·5	4·	4·5	5·	6	7
·00005					·62	·67	·70	·74	·77	·79	·81	·85	·88
·00007					·63	·67	·71	·73	·76	·78	·80	·84	·87
·0001	·40	·47	·52	·58	·64	·68	·71	·73	·76	·77	·79	·83	·86
·0002	·43	·49	·53	·60	·64	·68	·71	·73	·75	·77	·79	·81	·84
·0003	·44	·50	·54	·60	·65	·68	·71	·73	·75	·77	·78	·81	·83
·0004	·44	·50	·54	·60	·65	·68	·71	·73	·75	·76	·78	·80	·83
·0005	·44	·50	·54	·60	·65	.68	·71	·73	·75	·76	·78	·80	·82
·0008	·45	·51	·55	·61	·65	·69	·71	·73	·75	·76	·78	·80	·82
·001 and steeper falls.	·45	·51	·55	·61	·65	·69	·71	·73	·75	·76	·78	·80	·82

TABLE XII.—PART 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 025$, suitable to canals and rivers with earthen beds in perfect order and regimen, and perfectly free from stones and weeds—*continued.* (Kutter.)

For values of S.	For values of R in feet. 8	9	10	11	12	13	14	15	16	17	18	19	20
·00002					1·10	1·10	1·15	1·17	1·19	1·21	1·23	1·25	1·27
·00003					1·05	1·07	1·08	1·10	1·12	1·14	1·15	1·17	1·18
·00005	·91	·93	·96	·98	1·00	1·01	1·03	1·04	1·06	1·08	1·09	1·09	1·10
·00007	·89	·91	·93	·95	·97	·98	·99	1·00	1·01	1·03	1·04	1·05	1·05
·0001	·88	·90	·91	·93	·94	·95	·97	·97	·99	1·00	1·00	1·01	1·02
·0002	·85	·87	·89	·90	·91	·92	·93	·94	·95	·95	·96	·97	·97
·0003	·85	·87	·88	·89	·90	·91	·92	·92	·93	·94	·95	·95	·95
·0004	·85	·86	·87	·88	·89	·91	·91	·92	·93	·93	·94	·94	·95
·0005	·84	·86	·87	·88	·89	·90	·90	·91	·92	·93	·93	·94	·94
·0008	·84	·85	·87	·87	·88	·89	·90	·91	·91	·92	·92	·93	·93
·001 and steeper falls.	·84	·85	·86	·87	·88	·89	·90	·91	·91	·92	·92	·93	·93

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient $f = \cdot 030$, suitable to rivers and canals with earthen beds in moderately good order and regimen, having stones and weeds occasionally. (Kutter.)

For values of S.	For values of R in feet.												
	·5	·75	1	1·5	2	2·5	3	3·5	4	4·5	5	6	7
·00005				·47	·51	·55	·59	·62	·64	·66	·68	·72	·75
·00007				·47	·52	·56	·59	·61	·64	·66	·68	·71	·73
·0001	·32	·38	·42	·48	·53	·56	·59	·61	·64	·65	·67	·70	·72
·0002	·34	·39	·43	·49	·53	·56	·59	·61	·63	·65	·66	·69	·71
·0003	·35	·40	·44	·50	·54	·56	·59	·61	·63	·64	·66	·68	·71
·0004	·35	·40	·44	·50	·54	·56	·59	·61	·63	·64	·65	·68	·70
·0005	·35	·41	·44	·50	·54	·56	·59	·61	·63	·64	·65	·68	·70
·0008	·36	·41	·45	·50	·54	·57	·59	·61	·63	·64	·65	·68	·70
·001 and steeper falls.	·36	·41	·45	·50	·54	·57	·59	·61	·63	·64	·65	·68	·70

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient of $f = \cdot 030$—*continued.*
(Kutter.)

For values of S.	For values of R in feet. 8	9	10	11	12	13	14	15	16	17	18	19	20
·00002					·94	·96	·99	1·01	1·03	1·05	1·06	1·08	1·09
·00003					·90	·92	·94	·96	·97	·99	1·00	1·02	1·03
·00005	·78	·80	·82	·84	·85	·87	·88	·90	·91	·92	·94	·95	·95
·00007	·76	·78	·80	·82	·83	·84	·86	·87	·88	·89	·90	·91	·92
·0001	·74	·76	·78	·79	·81	·82	·83	·84	·85	·86	·87	·88	·88
·0002	·73	·74	·76	·77	·78	·79	·80	·81	·81	·82	·83	·85	·85
·0003	·72	·74	·75	·76	·77	·78	·79	·80	·80	·81	·82	·82	·83
·0004	·72	·73	·74	·75	·77	·77	·78	·79	·80	·80	·81	·82	·82
·0005	·72	·73	·74	·75	·76	·77	·78	·79	·79	·80	·80	·81	·82
·0008	·71	·73	·73	·74	·75	·76	·77	·78	·78	·79	·79	·80	·80
·001 and steeper falls	·71	·73	·73	·74	·75	·76	·77	·78	·78	·79	·79	·80	·80

TABLE XII.—Part 3—*continued.*

Co-efficients of velocity of discharge for surfaces having a frictional co-efficient f = ·035, suitable to rivers and canals with earthen beds in bad order and regimen, having stones and weeds in great quantities. (Kutter.)

For values of S.	For values of R in feet.											
	·5	·75	1	2	3	4	5	6	7	8	9	10
·00005				·44	·50	·55	·59	·62	·65	·68	·70	·72
·00007				·44	·50	·55	·58	·62	·64	·66	·68	·70
·0001	·27	·32	·35	·45	·50	·55	·58	·61	·63	·65	·67	·69
·0002	·28	·33	·36	·45	·51	·54	·57	·60	·62	·64	·65	·66
·0003	·29	·33	·37	·45	·51	·54	·57	·59	·61	·63	·64	·66
·0004	·29	·34	·37	·45	·51	·54	·57	·59	·61	·63	·64	·65
·0005	·29	·34	·37	·46	·51	·54	·57	·59	·61	·62	·64	·65
·0008	·30	·35	·37	·46	·51	·54	·57	·59	·61	·62	·63	·64
·001 and steeper falls	·30	·35	·37	·46	·51	·54	·57	·59	·61	·62	·63	·64

TABLE XII.—Part 4.

Co-efficients of Discharge for Orifices, being values of m for the formula in Table IX., and given in the Text.

$$V = m \times 8{\cdot}025 \sqrt{H}$$

Applied in the Table.	According to Experiment.	
·55	·572	Rectangular, length 7 depth, (L 7 D); *see* next page.
·6	·709	
·62	·62	Orifices generally.
·66	·66	Sluices without side walls.
·7	·7	Canal lock gates and dock gates.
·727	·62	Undershot wheel gates.
·8	·8	Velocity of approach in a channel.
·84	·83	Sluices in lock gates.
·84	·84	Large vertical pipes.
·9	·9	Narrow bridge openings.
·96	·94	Large sluices.
·96	·96	Wide openings from reservoirs.
·96	·96	Wide bridge openings.
·96	·96	Orifices with converging mouth-pieces.
1·	1·	Large orifices with diverging mouth-pieces.
	1·3	Attached diverging mill channels.

Modification of the co-efficient m, so as to include the effect due to velocity of approach;

Let h = head due to this velocity only,

then $m^1 = m \sqrt{1 + \frac{h}{H}}$

and m^1 is the new co-efficient to be used.

TABLE XII.—Part 4—*continued.*

Co-efficients of Discharge for Orifices—*continued.*

Table of co-efficients of Velocity of Discharge for Rectangular Orifices, when the depth (D) is less than the width (W) for a head (H).

$\frac{H}{W}$	$\frac{D}{W}$ = ·1	$\frac{D}{W}$ = ·5	$\frac{D}{W}$ = ·25	$\frac{D}{W}$ = ·15	$\frac{D}{W}$ = ·1	$\frac{D}{W}$ = ·05
	Values of M.					
·05						·709
·10					·660	·698
·15				·638	·660	·691
·20			·612	·640	·659	·685
·25			·617	·640	·659	·682
·30			·622	·640	·658	·678
·40		·600	·626	·639	·657	·671
·50		·605	·628	·638	·655	·667
·60	·572	·609	·630	·637	·654	·664
·75	·585	·611	·631	·635	·653	·660
1·00	·592	·613	·634	·634	·650	·655
1·50	·598	·616	·632	.632	·645	·650
2·00	·400	·617	·631	·631	·642	·647
2·50	·602	·617	·631	·630	·640	·643
3·50	·604	·616	·629	·629	·637	·638
4·00	·605	·615	·627	·627	·632	·627
6·00	·604	·613	·623	·623	·625	·621
8·00	·602	·611	·619	·619	·618	·616
10·00	·601	·607	·613	·613	·613	·613

The above was deduced by Rankine from results of experiments by Poncelet and Lesbros.

N.B.—When H 7 3 D, the centre of figure may be considered the centre of motion.

TABLE XII.—Part 5.

*Co-efficients of Discharge for Overfalls, being values of *m* for the formula applied in Table IX., and given in the Text.

$$V = \tfrac{2}{3}\, m \times 8{\cdot}025 \sqrt{H}$$

N.B. 1 = length of weir sill : L = length of dam, or breadth of channel : H = head on sill : D = depth of notch.

In Table.	By Experimentalists.	
·5	·5	Broad-crested or flat-topped dams Dams with a channel attached
·55	·595 ·662	Weirs with 1-inch crests when $1 =$ or $> \frac{L}{4}$; the exact value of m being $= {\cdot}57 \times \frac{1}{10L}$
·6	·6	Overfalls when $1 > \frac{L}{4}$ and $< \frac{L}{3}$ V-shaped notch, when $1 = \frac{D}{2}$
·62	·26	V-shaped notch, when $1 = \frac{D}{4}$
·666	·552	Weirs when $1 \doteq L$, and $H > \frac{1}{3}$ height of the barrier; in this case the velocity of approach must be considered in addition.
·7 ·727	·666	Weirs generally when $1 = L$ and $H < \frac{1}{3}$ the height of the barrier.

To modify the co-efficient *m* so as to include the effect due to velocity of approach,

Let h = head due to velocity of approach only ;—

$$\text{then } m^1 = m \left\{ \left(1 \times \frac{h}{H}\right)^{\frac{3}{2}} - \left(\frac{h}{H}\right)^{\frac{3}{2}} \right\}$$

and m^1 is the new co-efficient to be used.

---

* In using Table VIII. for overfalls, always diminish the velocity of discharge there given by one-third ; this alone admits of the use of the same table for discharges both of orifices and overfalls.

TABLE XII—Part 6.

Hydraulic Memoranda.

Measures.

Feet	×	·015	= Gunter's chains.
Feet	×	·00019	= Miles.
Square feet	×	·111	= Square yards.
Square feet	×	·000023	= Acres.
Cubic feet	×	6·23	= Gallons.
Cubic feet	×	·779	= Bushels.
Cubic feet	×	·037	= Cubic yards.

Rainfall.

Feet of downpour	×	193600	= cubic feet per square mile.
Feet of downpour	×	302·5	= cubic feet per acre.

Drainage areas.

The drainage from 1 square mile collecting 1 foot yearly } will irrigate 176 acres at a duty of 200 acres, will supply 47,580 inhabitants at a duty of 10 gallons daily, will yield ·8833 cubic feet per second throughout the year.

Velocities.

Feet per second	×	·68	give miles per hour.
Feet per second	×	60	give feet per minute.
Feet per second	×	20	give yards per minute.
Feet per second	×	1200	give yards per hour.

Discharges.

Cub. feet per sec.	×	2·2	give cubic yards per minute.
Cub. feet per sec.	×	133	give cubic yards per hour.
Cub. feet per sec.	×	3200	give cubic yards per day.
Cub. feet per sec.	×	6¼	give gallons per second.
Cub. feet per sec.	×	375	give gallons per minute.
Cub. feet per sec.	×	22	give thousands of gallons per hour.
Cub. feet per sec.	×	500	give thousands of gallons per day.
Cub. feet per sec.	×	2400	give tons per day.

TABLE XII.—Part 6—*continued.*

Cubic feet.		Gallons.	
1·	=	6·232	and weighs 62·32 lbs.
·1605	=	1	and weighs 10 lbs.
1·8	=	11·2	and weighs 1 cwt.
35·943	=	224	and weighs 1 ton.
1 cubic inch	=	·0036	and weighs ·0361 lbs.

1 fluid ounce weighs 437·5 grains.
1 Troy ounce measures 8 fluid ounces, 46 minims.
1 Avoirdupois ounce measures 8 fluid ounces.
1 lb. Troy =5760 grains =6319·54 minims of water.
1 gallon =76800 minims =70000 grs. of distilled water.
1 lb. Avoir. =7000 grains.

All comparisons between measures of capacity and those of weight are made with distilled water at a specific gravity of 1, temp. 62°.

Pressure.

H = head of water in feet $H = P \times 2{\cdot}31$.
P = pressure in lbs. per square foot $P = H \times 62{\cdot}32$.

Horse-power.

1 HP = 33000 lbs. raised 1 foot in 1 minute.
= 884 tons raised 1 foot in 1 hour.

Theoretical HP = ·113 Q × fall in feet.

The drainage of 10 square miles collecting 12″ yearly gives 1 HP for each foot of fall.

For pumping engines of the best class, allow HP = ·142 Q H where Q = quantity raised in cubic feet per second, H = height in feet.

Mills.—An ordinary mill will grind 1 bushel per HP per hour; for each pair of stones allow 4 HP nominal.

Towage.

The general formula referred to in the text is

$$R = b\, T.\, V^2,$$

where R = the pull on the rope in pounds,
T = the displacement of the barge in tons,
V = the velocity through the water,
b = a co-efficient varying with the form of the barge, from ·109 to ·369.

TABLE XII.—Part 7.

Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area of Circle.	Square.	Square Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
·01	·031	·00008	·0001	·1	·2154	·3981	·00001	·1584	100·	$\bar{2}$·
·015	·047	·00018	·0002	·1225	·2466	·4317	·00003	·1864	66·66	$\bar{2}$·1760913
·02	·063	·0003	·0004	·1414	·2714	·4573	·00006	·2140	50·	$\bar{2}$·3010300
·025	·078	·0005	·0006	·1581	·2924	·4782	·00010	·2287	40·	$\bar{2}$·3979400
·03	·094	·0007	·0009	·1732	·3107	·4959	·00017	·2460	33·33	$\bar{2}$·4771213
·035	·110	·0010	·0012	·1871	·3271	·5115	·00023	·2616	28·57	$\bar{2}$·5440680
·04	·126	·0013	·0016	·2	·3420	·5253	·00032	·2759	25·	$\bar{2}$·6020600
·045	·141	·0016	·0020	·2121	·3557	·5378	·00043	·2893	22·22	$\bar{2}$·6532125
·05	·157	·0020	·0025	·2236	·3684	·5493	·00056	·3017	20·	$\bar{2}$·6989700
·055	·173	·0024	·0030	·2345	·3803	·5599	·00071	·3134	18·18	$\bar{2}$·7403627
·06	·188	·0028	·0036	·2449	·3915	·5697	·00088	·3245	16·67	$\bar{2}$·7781513
·065	·204	·0033	·0042	·2550	·4021	.5789	·00108	·3351	15·38	$\bar{2}$·8129134
·07	·220	·0038	·0049	·2646	·4121	·5875	·00130	·3452	14·29	$\bar{2}$·8450980
·075	·236	·0044	·0056	·2739	·4217	·5957	·00154	·3548	13·33	$\bar{2}$ 8750613
·08	·251	·0050	·0064	·2828	·4309	·6034	·00181	·3641	12·5	$\bar{2}$·9030900
·085	·267	·0057	·0072	·2915	·4397	·6108	·00211	·3731	11·76	$\bar{2}$·9294189
.09	·283	·0064	·0081	·3	·4481	·6178	·00243	·3817	11·11	$\bar{2}$·9542425
·095	·298	·0071	·0090	·3082	·4563	·6245	·00278	·3900	10·53	$\bar{2}$·9777236

TABLE XII.—Part 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
·1	·314	·0078	·01	·3162	·4642	·6310	·0032	·3981	10·	$\bar{1}$·
·15	·471	·0177	·0225	·3873	·5313	·6843	·0087	·4682	6·666	$\bar{1}$·1760913
·2	·628	·0314	·04	·4472	·5848	·7248	·0179	·5253	5·	$\bar{1}$·3010300
·25	·785	·0491	·0625	·5	·6300	·7579	·0313	·5743	4·	$\bar{1}$·3979400
3	·942	·0706	·09	·5477	·6694	·7860	·0493	·6178	3·333	$\bar{1}$·4771213
·35	1·100	·0962	·1225	·5916	·7047	·8106	·0769	·6571	2·857	$\bar{1}$·5440680
·4	1·257	·1256	·1600	·6324	·7368	·8326	·1012	·6931	2·5	$\bar{1}$·6020600
·45	1·414	·1590	·2025	·6708	·7663	·8524	·1358	·7266	2·222	$\bar{1}$·6532125
·5	1·571	·1963	·25	·7071	·7937	·8706	·1768	·7579	·2·	$\bar{1}$·6989700
·52	1·634	·2124	·2704	·7211	·8041	·8774	·1950	·7698	1·924	$\bar{1}$·7160033
·54	1·696	·2290	·2916	·7348	·8143	·8841	·2143	·7816	1·852	$\bar{1}$·7323938
·56	1·759	·2463	·3136	·7483	·8243	·8905	·2347	·7930	1·786	$\bar{1}$·7481880
·58	1·822	·2642	·3364	·7616	·8340	·8968	·2562	·8042	1·724	$\bar{1}$·7634280
·6	1·884	·2827	·36	·7746	·8434	·9029	·2788	·8152	1·667	$\bar{1}$·7781513
·62	1·948	·3019	·3844	·7874	·8527	·9088	·3027	·8260	1·613	$\bar{1}$·7923917
·64	2·011	·3217	·4096	·8	·8618	·9146	·3277	·8365	1·563	$\bar{1}$·8061800
·66	2·073	·3421	·4356	·8124	·8707	·9203	·3539	·8469	1·515	$\bar{1}$·8195439
·68	2·136	·3632	·4624	·8246	·8794	·9258	·3813	·8570	1·471	$\bar{1}$·8325089
·7	2·199	·3848	·49	·8366	·8879	·9312	·4100	·8670	1·429	$\bar{1}$·8450980
·72	2·262	·4072	·5184	·8485	·8963	·9364	·4399	·8769	1·389	$\bar{1}$·8573325
·74	2·325	·4301	·5476	·8602	·9045	·9416	·4711	·8865	1·351	$\bar{1}$·8692317
·76	2·388	·4537	·5776	·8718	·9126	·9466	·5036	·8961	1·316	$\bar{1}$·8808316
·78	2·450	·4778	·6084	·8832	·9205	·9515	·5373	·9054	1·282	$\bar{1}$·8920946
·8	2·513	·5026	·64	·8944	·9283	·9564	·5724	·9146	1·250	$\bar{1}$·9030900
·82	2·576	·5281	·6724	·9055	·9360	·9611	·6089	·9237	1·220	$\bar{1}$·9138139

TABLE XII.—Part 7—*continued*. Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
·84	2·639	·5542	·7056	·9165	·9435	·9657	·6467	·9327	1·190	$\bar{1}$·9242793
·86	2·702	·5809	·7396	·9274	·9510	·9702	·6859	·9415	1·163	$\bar{1}$·9344985
·88	2·765	·6082	·7744	·9381	·9583	·9748	·7265	·9502	1·136	$\bar{1}$·9444827
·9	2·827	·6362	·81	·9487	·9655	·9791	·7684	·9587	1·111	$\bar{1}$·9542425
·92	2·890	·6648	·8464	·9592	·9726	·9834	·8118	·9672	1·087	$\bar{1}$·9637878
·94	2·953	·6940	·8836	·9695	·9796	·9877	·8567	·9756	1·064	$\bar{1}$·9731279
·96	3·016	·7238	·9216	·9798	·9865	·9918	·9030	·9838	1·042	$\bar{1}$·9822712
·98	3·079	·7543	·9604	·9899	·9933	·9960	·9507	·9920	1·020	$\bar{1}$·9912261
1·	3·141	·7854	1·	1·	1·	1·	1.	1·	1·	0·
1·25	3·927	1·2272	1·5625	1·1180	1·0772	1·0456	1·7469	1·0934	·8	0·0969100
1·5	4·712	1·7671	2·25	1·2247	1·1447	1·0845	2·7556	1·1761	·6666	0·1760913
1·75	5·947	2·4053	3·0625	1·3229	1·2051	1·1184	4·0513	1·2500	·5714	0·2430880
2·	6·283	3·1416	4·	1·4142	1·2599	1·1487	5·6569	1·3195	·5	0·3010300
2·25	7·068	3·9761	5·0625	1·5	1·3104	1·1761	7·5937	1·3882	·4444	0·3521825
2·5	7·854	4·9087	6·25	1·5811	1·3572	1·2011	9·8823	1·4427	·4	0·3979400
2·75	8·639	5·9396	7·5625	1·6583	1·4010	1·2242	12·541	1·4988	·3636	0·4393327
3·	9·425	7·0686	9·	1·7321	1·4423	1·2457	15·589	1·5518	·3333	0·4771213
3·25	10·21	8·2958	10·5625	1·8028	1·4812	1·2658	19·041	1·6023	·3077	0·5118834
3·5	10·99	9·6211	12·25	1·8708	1·5183	1·2846	22·918	1·6505	·2857	0·5440680
3·75	11·78	11·0469	14·0625	1·9365	1·5536	1·3026	27·232	1·6967	·2666	0·5740313
4·	12·57	12·5664	16·	2·	1·5874	1·3195	32·	1·7411	·25	0·6020600
4·25	13·35	14·1863	18·0625	2·0616	1·6198	1·3356	37·2861	1·7838	·2353	0·6283889
4·5	14·13	15·9043	20·25	2·1213	1·6510	1·3510	42·9561	1·8251	·2222	0·6532125
4·75	14·92	17·7206	22·5625	2·1794	1·6810	1·3656	49·1781	1·8650	·2105	0·6766936
5·	15·71	19·6350	25·	2·2361	1·7099	1·3804	55·9010	1·9054	·2	0·6989700

TABLE XII.—PART 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
5·5	17·27	23·7583	30·25	2·3452	1·7652	1·4063	70·94	1·9776	·1818	0·7403627
6·	18·84	28·2744	36·	2·4495	1·8171	1·4310	88·18	2·0477	·1667	0·7781513
6·5	20·42	33·1831	42·25	2·5495	1·8663	1·4541	107·71	2·1143	·1538	0·8129134
7·	21·99	38·4846	49·	2·6458	1·9129	1·4758	129·64	2·1779	·1429	0·8450980
7·5	23·56	44·1787	56·25	2·7386	1·9574	1·4963	154·04	2·2388	·1333	0·8750613
8·	25·13	50·2656	64·	2·8284	2·	1·5157	181·01	2·2974	·1250	0·9030900
8·5	26·70	56·7451	72·25	2·9155	2·0408	1·5342	214·64	2·3538	·1176	0·9294189
9·	28·27	63·6174	81·	3·	2·0801	1·5518	243·	2·4082	·1111	0·9542425
9·5	29·84	70·8823	90·25	3·0822	2·1179	1·5687	278·16	2·4609	·1053	0·9777236
10·	31·41	78·5400	100·	3·1623	2·1544	1·5849	316·23	2·5119	·1	1·
11	34·55	95·033	121	3·3166	2·2239	1·6154	401·31	2·6095	·0909	1·0413927
12	37·69	113·097	144	3·4641	2·2894	1·6437	498·83	2·7019	·0833	1·0791812
13	40·84	132·732	169	3·6056	2·3513	1·6702	609·34	2·7896	·0769	1·1130494
14	43·98	153·938	196	3·7417	2·4101	1·6952	733·36	2·8738	·0714	1·1461280
15	47·12	176·715	225	3·8729	2·4662	1·7188	871·42	2·9543	·0667	1·1760913
16	50·26	201·062	256	4·	2·5198	1·7411	1024·	3·0314	·0625	1·2041200
17	53·40	226·980	289	4·1231	2·5713	1·7623	1191·8	3·1058	·0588	1·2304489
18	56·54	254·469	324	4·2426	2·6207	1·7826	1374·6	3·1777	·0556	1·2552725
19	59·69	283·529	361	4·3589	2·6684	1·8020	1573·5	3·2472	·0526	1·2787536
20	62·83	314·160	400	4·4721	2·7144	1·8206	1788·8	3·3145	·0500	1·3010300
21	65·97	346·361	441	4·5826	2·7589	1·8384	2020·9	3·3798	·0476	1·3222193
22	69·11	380·133	484	4·6904	2·8020	1·8556	2270·11	3·4438	·0454	1·3424227
23	72·25	415·476	529	4·7958	2·8439	1·8722	2537·00	3·5050	·0435	1·3617278
24	75·39	452·390	576	4·8989	2·8845	1·8882	2821·8	3·5652	·0417	1·3802112
25	78·54	490·875	625	5·	2·9240	1·9037	3125·0	3·6239	·0400	1·3979400

TABLE XII.—Part 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circum-ference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
26	81·68	530·930	676	5·0990	2·9625	1·9186	3446·9	3·6812	·0385	1·4149733
27	84·82	572·556	729	5·1962	3·	1·9332	3788·0	3·7372	·0370	1·4313638
28	87·96	615·753	784	5·2915	3·0366	1·9473	4148·5	3·7920	·0357	1·4471580
29	91·10	660·521	841	5·3852	3·0723	1·9610	4528·9	3·8455	·0345	1·4623980
30	94·24	706·860	900	5·4772	3·1072	1·9744	4929·5	3·8981	·0333	1·4771213
31	97·38	754·769	961	5·5678	3·1414	1·9873	5350·6	3·9493	·0323	1·4913617
32	100·5	804·249	1024	5·6569	3·1748	2·	5792·6	4·	·0313	1·5051500
33	103·6	855·300	1089	5·7746	3·2075	2·0124	6255·8	4·0495	·0303	1·5185139
34	106·8	907·922	1156	5·8309	3·2396	2·0244	6740·5	4·0982	·0294	1·5314789
35	109·9	962·115	1225	5·9161	3·2711	2·0362	7247·2	4·1460	·0285	1·5440680
36	113·0	1017·87	1296	6·	3·3019	2·0477	7776·0	4·1930	·0278	1·5563025
37	116·2	1075·21	1369	6·0828	3·3322	2·0589	8327·3	4·2392	·0270	1·5682017
38	119·3	1134·11	1444	6·1644	3·3620	2·0699	8901·4	4·2846	·0263	1·5797836
39	122·5	1194·59	1521	6·2449	3·3912	2·0807	9498·6	4·3294	·0256	1·5910646
40	125·6	1256·64	1600	6·3245	3·4200	2·0913	10120	4·3735	·0250	1·6020600
41	128·8	1320·25	1681	6·4031	3·4482	2·1017	10763	4·4169	·0244	1·6127839
42	131·9	1385·44	1764	6·4807	3·4760	2·1118	11432	4·4596	·0238	1·6232493
43	135·0	1452.20	1849	6·5574	3·5034	2·1217	12124	4·5018	·0233	1·6334685
44	138·2	1520·53	1936	6·6332	3·5303	2·1315	12842	4·5434	·0227	1·6434527
45	141·3	1590·43	2025	6·7082	3·5569	2·1411	13584	4·5844	·0222	1·6532125
46	144·5	1661·90	2116	6·7823	3·5830	2·1506	14351	4·6249	·0217	1·6627578
47	147·6	1734·94	2209	6·8557	3·6088	2·1598	15144	4·6649	·0213	1·6720979
48	150·7	1809·56	2304	6·9282	3·6342	2·1689	15962	4·7043	·0208	1·6812412
49	153·9	1885·74	2401	7·	3·6593	2·1779	16807	4·7433	·0204	1·6901961
50	157·0	1963·50	2500	7·0711	3·6840	2·1867	17677	4·7818	·0200	1·6989700

TABLE XII.—Part 7—*continued*. Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
51	160·2	2042·	2601	7·1414	3·7084	2·1954	18574	4·8198	·01961	1·7075702
52	163·3	2123·	2704	7·2111	3·7325	2·2039	19499	4·8574	·01924	1·7160033
53	166·5	2206·	2809	7·2801	3·7563	2·2124	20449	4·8945	·01887	1·7242759
54	169·6	2290·	2916	7·3484	3·7798	2·2206	21428	4·9313	·01852	1·7323938
55	172·7	2375·	3025	7·4162	3·8030	2·2288	22435	4·9676	·01818	1·7403627
56	175·9	2463·	3136	7·4833	3·8259	2·2369	23468	5·0035	·01786	1·7481880
57	179·0	2551·	3249	7·5498	3·8485	2·2448	24529	5·0391	·01754	1·7558749
58	182·2	2642·	3364	7·6158	3·8709	2·2526	25619	5·0742	·01724	1·7634280
59	185·3	2733·	3481	7·6811	3·8930	2·2603	26638	5·1091	·01695	1·7708520
60	188·4	2827·	3600	7·7460	3·9149	2·2679	27885	5·1435	·01667	1·7781513
61	191·6	2922·	3721	7·8102	3·9365	2·2754	29061	5·1776	·01639	1·7853298
62	194·7	3019·	3844	7·8740	3·9579	2·2829	30267	5·2114	·01613	1·7923917
63	197·9	3117·	3969	7·9372	3·9791	2·2902	31503	5·2449	·01587	1·7993405
64	201·0	3216·	4096	8·	4·	2·2974	32768	5·2780	·01563	1·8061800
65	204·2	3318·	4225	8·0623	4·0207	2·3045	34063	5·3109	·01538	1·8129134
66	207·3	3421·	4356	8·1240	4·0412	2·3116	35388	5·3434	·01515	1·8195439
67	210·4	3525·	4489	8·1854	4·0615	2·3185	36744	5·3756	·01493	1·8260748
68	213·6	3631·	4624	8·2462	4·0817	2·3254	38130	5·4076	·01471	1·8325089
69	216·7	3739·	4761	8·3066	4·1016	2·3322	39547	5·4393	·01449	1·8388491
70	219·9	3848·	4900	8·3666	4·1213	2·3389	40996	5·4707	·01429	1·8450980
71	223·0	3959·	5041	8·4261	4·1408	2·3456	42476	5·5018	·01408	1·8512583
72	226·1	4071·	5184	8·4853	4·1602	2·3522	43987	5·5326	·01389	1·8573325
73	229·3	4185·	5329	8·5440	4·1793	2·3588	45531	5·5639	·01370	1·8633229
74	232·4	4300·	5476	8·6023	4·1983	2·3651	47106	5·5936	·01351	1·8692317
75	235·6	4417·	5625	8·6603	4·2172	2·3714	48714	5·6237	·01333	1·8750613

TABLE XII.—Part 7—*continued.* Useful Numbers, Powers and Roots.

Number, Diameter, or Head.	Circumference.	Area.	Square.	Sq. Root.	Cube Root.	Fifth Root.	$\sqrt[2]{d^5}$	$\sqrt[5]{d^2}$	Reciprocal.	Logarithm.
76	238·7	4536·	5776	8·7178	4·2358	2·3777	50354	5·6536	·01316	1·8808136
77	241·9	4656·	5929	8·7750	4·2543	2·3840	52026	5·6832	·01299	1·8864907
78	245·0	4778·	6084	8·8318	4·2727	2·3901	53732	5·7127	·01282	1·8920946
79	248·1	4901·	6241	8·8882	4·2908	2·3962	55471	5·7418	·01266	1·8976271
80	251·3	5026·	6400	8·9443	4·3089	2·4022	57243	5·7708	·01250	1·9030900
81	254·4	5153·	6561	9·	4·3267	2·4082	59049	5·7995	·01235	1·9084850
82	257·6	5281·	6724	9·0554	4·3445	2·4141	60888	5·8281	·01220	1·9138139
83	260·7	5410·	6889	9·1104	4·3621	2·4200	62762	5·8564	·01205	1·9190781
84	263·8	5541·	7056	9·1652	4·3795	2·4258	64669	5·8845	·01190	1·9242793
85	267·0	5674·	7225	9·2195	4·3968	2·4315	66611	5·9125	·01170	1·9294189
86	270·1	5808·	7396	9·2736	4·4140	2·4372	68589	5·9402	·01163	1·9344985
87	273·3	5944·	7569	9·3273	4·4310	2·4429	70559	5·9677	·01149	1·9395193
88	276·4	6082·	7744	9·3808	4·4480	2·4485	72645	5·9951	·01136	1·9444827
89	279·6	6221·	7921	9·4340	4·4647	2·4540	74726	6·0222	·01124	1·9493900
90	282·7	6361·	8100	9·4868	4·4814	2·4595	76843	6·0492	·01111	1·9542425
91	285·8	6503·	8281	9·5394	4·4979	2·4650	78995	6·0760	·01099	1·9590414
92	289·0	6647·	8464	9·5917	4·5144	2·4703	81183	6·1026	·01087	1·9637878
93	292·1	6792·	8649	9·6437	4·5307	2·4757	83408	6·1291	·01075	1·9684829
94	295·3	6939·	8836	9·6954	4·5468	2·4810	85668	6·1553	·01064	1·9731279
95	298·4	7088·	9025	9·7468	4·5629	2·4863	87964	6·1814	·01033	1·9777236
96	301·5	7238·	9216	9·7980	4·5789	2·4915	90298	6·2074	·01042	1·9822712
97	304·7	7389·	9409	9·8489	4·5947	2·4966	92668	6·2332	·01031	1·9867717
98	307·8	7542·	9604	9·8995	4·6104	2·5018	95075	6·2588	·01020	1·9912261
99	311·0	7697·	9801	9·9499	4·6261	2·5068	97519	6·2843	·01010	1·9956352
100	314·1	7854·	10000	1·0	4·6416	2·5119	100000	6·3096	·01000	2·0

N.B.—This table may also be used for finding the fourth and fifth powers of numbers.

APPENDIX

OF

MISCELLANEOUS TABLES AND DATA.

Retaining Walls.
Masonry Dams.
Thickness and Weight of Pipes.
Duty of Hydraulic Machines.
Indian Hydraulic Contrivances.
Constants of Labour and Cartage.

MISCELLANEOUS TABLES AND DATA.

Formulæ and Data for Retaining Walls.

Extracted from various articles by J. H. E. Hart, Esq., C.E.

General equation for breadth of base, $x = \sqrt{\left\{\frac{n\,H}{3\,w\,(q \pm q^1)}\right\}}$

Where H = total horizontal pressure against the back of the wall.

n = the ratio of its sectional area to that of a rectangle of equal height and breadth.

w = the weight of a cubic foot of the wall.

qx = the horizontal deviation of the centre of resistance of the base from the middle of the base.

q^1x = the horizontal deviation of the centre of gravity of the profile from the middle of the base.

(1) For vertical rectangular sections, $n = 1, q^1 = 0, x = \sqrt{\left(\frac{H}{3wq}\right)}$

(2) For plumb-faced trapezoidal sections of a top thickness (t)

$$n = \frac{x + t}{2\,x} \text{ and } q^1 = \left(\frac{t - x}{6}\right)\left(\frac{x + 2\,t}{x\,(x + t)}\right)$$

$$x = \sqrt{\left\{\frac{2\,H - w\,t^2}{3\,w\,(q - \frac{1}{6})} + \left(\frac{t}{2}\right)^2\right\}} - \frac{t}{2}$$

(3) For plumb-backed trapezoidal sections of a top thickness (t)

$$n = \frac{x + t}{2\,x} \text{ and } q^1 = \left(\frac{x - t}{6}\right)\left(\frac{x + 2\,t}{x\,(x + t)}\right)$$

$$x = \sqrt{\left\{\frac{2\,H + w\,t^2}{3\,w\,(q + \frac{1}{6})} + \left(\frac{t}{2}\right)^2\right\}} - \frac{t}{2}$$

The limiting value of q to avoid the existence of tension in the masonry is $\frac{1}{6}$, but its limiting value in actual practice is $\frac{1}{4}$. In special cases, since it must not be so great as to cause the maximum pressure (P) to exceed the safe resistance (C) to crushing of the material, its values correspond as follows to the values of $\frac{P}{p}$, where p = the mean pressure per unit of surface of base, = sum of the vertical forces ÷ area of the base; and P is less than C.

$$q = \tfrac{1}{12}, \tfrac{1}{11}, \tfrac{1}{10}, \tfrac{1}{9}, \tfrac{1}{8}, \tfrac{1}{7}, \tfrac{1}{6};\ \tfrac{2}{11}, \tfrac{1}{5}, \tfrac{2}{9}, \tfrac{1}{4}$$

$$\frac{P}{p} = \tfrac{3}{2}, \tfrac{17}{11}, \tfrac{8}{5}, \tfrac{5}{3}, \tfrac{7}{4}, \tfrac{13}{7}, 2;\ \tfrac{15}{7}, \tfrac{20}{9}, \tfrac{12}{5}, \tfrac{8}{3}$$

If x = thickness of a vertical rectangular wall to sustain a horizontal-topped bank,

x_1 = do. for an indefinite surcharge,

x_2 = do. for a surcharge of a height c,

$$x_2 = \frac{h\,x + 2\,c\,x_1}{h + 2\,c} \text{ where } h = \text{height of the wall.}$$

h

MISCELLANEOUS DATA—*continued.*

Co-efficients for Earth Pressure against one foot in length of vertical-backed Walls for various angles of repose of earth.

For angles of repose of	27°	30°	33°	36°	39°	42°	45°	48°
	Co-efficients of earth pressure.							
Earth horizontal at the level of the top ...	·188	·167	·147	·130	·114	·099	·085	·073
Indefinite Surcharge to angle of repose ...	·397	·375	·351	·327	·302	·276	·250	·224

Horizontal pressure H = co-efficient × weight of 1 cubic foot × h^2.

For walls having sloping backs the horizontal pressure is conveniently determined by Neville's well-known geometrical method, which gives the position of the plane of maximum pressure, and hence also the values of e the inclination of that plane with the angle of repose, and A the sectional area of effective pressure, in the general expression for horizontal thrust, H = A tan e × weight of 1 cubic foot of the earth.

For water pressure $H = 31{\cdot}2 \times h^2$.

Working Loads or safe units of pressure adopted in existing structures.

(From Spon's "Dictionary of Engineering.")

	Tons on the square foot.
Soft rock foundations	9
Concrete	3
Earth	1¼
Ashlar masonry, limestone, Britannia Bridge	16
„ „ granite, Saltash Bridge	10
„ backed with rubble, Peniston Viaduct	6
Rubble masonry, sandstone in Aberthaw lime, Pont y Pridd	20¾
„ „ limestone in chalk lime, Barentine Viaduct	3½
„ „ in hydraulic lime, Almanza Dam ...	12·8
„ „ „ „ Ban	7·3
„ „ „ „ Furens	6·
„ „ „ „ Tulsi	8·9 to 6·9
Brickwork, London paviors' in cement, Charing Cross Bridge	12
„ Staffordshire blue brick in cement, Clifton Suspension Bridge	10
„ red Birmingham in lias lime, Railway Viaduct	7
Cement mortar	20 to 32
Lime mortar	2½ to 5½

MISCELLANEOUS DATA—*continued.*

Table of Weights of Materials.

(From Spon's "Dictionary of Engineering.")

	Angle of repose.	Specific gravity.	Weight of a cubic foot.
Clay, dry	30° to 40°	1·95	120
„ wet	15 to 20	2·17	135
Earth, common dry	46	1·64	102
Earthy clay and sand	54	1·5 to 1·7	97 to 106
Gravel	37	1·5 to 1·9	96 to 120
Mould, garden	35 to 45	1·4	70 to 90
Sand, dry fine	34 to 40	1·4 to 1·6	84 to 97
„ damp	34 to 40	1·9	118
Shingle, loose	39	2·2	139
Basalts and traps		3 to 2·4	187 to 155
Bricks, red		2·16	135
„ common		1·76	110
„ stock (London)		1·84	115
Brickwork in cement		1·92	120
„ in new mortar		1·87	117
„ in old mortar		1·52	95
Cement new		1·61	100
Flint masonry		2·34	148
Granites		3·05 to 2·25	190 to 141
Granite masonry		2·75	172
Limestones		2·54 to 1·86	159 to 116
Mortars, new		1·9	119
„ old		1·42	89
Sandstones		2·67 to 1·38	168 to 88
Slates		2·9 to 2·5	180 to 157

Ashlar, weight $= \frac{7}{8}$ that of stone $+ \frac{1}{8}$ that of mortar.
Rubble, weight $= \frac{3}{4}$ to $\frac{2}{3}$ that of stone $+ \frac{1}{4}$ to $\frac{1}{3}$ that of mortar.

The safe working load for masonry and brickwork is that for the mortar used; but in ordinary calculation, 5 tons per square foot for brickwork and rubble, and 30 for ashlar in cement, is generally allowed.

MISCELLANEOUS DATA—*continued.*

Dimensions of Trapezoidal Masonry Dams, having both faces battering, for heights up to 40 *feet.* (By the Author.)

	Good rubble.	Inferior rubble.	Brickwork.
Height of dam ...	H	H	H
Thickness at top ...	$\frac{1}{8}$H	·2H	·3H
Thickness at bottom	·5H	·6H	·7H
Front batter ...	1 in 24	1 in 15	1 in 15
Back batter ...	1 in 3	1 in 3	1 in 3
Sectional area ...	·3H^2	·4H^2	·5H^2

Dimensions of Trapezoidal Masonry Dams, having the water face vertical, for heights up to 40 *feet.*

Weight of masonry per cubic foot	140 lbs.	120 lbs.	100 lbs.
Height of dam	H	H	H
Thickness at top	·24H	·25H	·28H
Thickness at bottom ...	·48H	·51H	·56H
Water face	Vertical	Vertical	Vertical
Outer face...	1 in 4·25	1 in 4	1 in 3·57
Sectional area	·36H^2	·375H^2	·42H^2
Weight per unit of length	50H^2	45H^2	42H^2
Mean pressure	104H	90H	75H
Maximum pressure ...	416H	360H	300H

These data apply to the same limiting value of q, the ratio to the breadth of the base of the distance along it from the foot at which the direction of the resultant pressure cuts, which is taken at one-third. A slight modification of the above section may be used for heights up to 50 feet. For lofty dams, the process and rules of Rankine for obtaining the dimensions of dams with curved profiles under different conditions yield correct results by means of short and simple calculations.

MISCELLANEOUS DATA—*continued.*

Thicknesses, Sizes, and Weights of Cast Iron Pipes. (Box.)

Safe Thickness for Various Pressures.

Diameter in inches.	Head of water in feet.					
	For Gas.	100	250	500	750	1000
	Thickness in inches.					
1½	·27	·28	·29	·30	·31	·33
2	·29	·3	·31	·33	·35	·37
2½	·3	·31	·33	·35	·37	·41
3	·32	·33	·35	·38	·41	·44
4	·35	·37	·39	·43	·47	·51
5	·37	·39	·42	·47	·52	·57
6	·39	·42	·[illegible]	·51	·57	·63
7	·41	·44	·48	·55	·62	·69
8	·43	·46	·51	·59	·67	·75
9	·45	·48	·53	·63	·72	·81
10	·46	·50	·56	·66	·76	·86
12	·49	·54	·61	·73	·85	·97
15	53	·59	·68	·83	·98	1·13
18	·57	·64	·75	·93	1·11	1·29
21	·6	·69	·81	1·02	1·23	1·44
24	·64	·73	·88	1·12	1·36	1·60
30	·69	·81	1·00	1 29	1·59	1·89
36	·75	·89	1·11	1 47	1·83	2·19

Sizes and Weights of Socket Pipes for a Head of 200 Feet.

Diameter.	Length without socket.	Depth of socket.	Lead joint, thick,	deep,	weight.	Average weight of pipe.		Average weight of quarter bends.	
inches	feet	inches	"	"	lbs.	cwt.	lbs.	cwt.	lbs.
1½	6	3	¼	1¼	1·2		42		28
2	6	3	¼	1½	1·4		56		30
2½	6	3¼	¼	1¾	1·6		67		37
3	9	3½	5/16	1¾	2·3	1	8		45
4	9	4	5/16	2	4·	1	56		76
[illegible]	9	4	5/16	2	5·	2			84
6	9	4¼	5/16	2¼	6·5	2	56		87
7	9	4¼	5/16	2¼	7·7	3	12	1	25
8	9	4¼	5/16	2¼	8·2	3	80	1	58
9	9	4⅜	5/16	2⅜	10·4	4	28	1	74
10	9	4⅜	5/16	2½	11·5	4	98	3	84
12	9	4½	5/16	2¾	15·	6	56	3	105

MISCELLANEOUS TABLES—*continued.*

Hydraulic Machines:—Return of Motive Power.

Deduced from Morin's Experiments.

	Proportion of Motive Power yielded.
Lift pump	·316
Force pump ...	·516
Fire engine ...	·233
Chinese wheel	·36
	·59
Flash wheel ...	·75
Wirtz pump	·181
	·640
ROTARY.	
Stotz pump ...	·43
Leclerc	·307
CENTRIFUGAL.	
Piatti	·20
Appold	·70
Gwynne ...	·190
	·300
Girard	·300
Vertical helix ...	·19
WATER RAMS.	
Montgolfier ...	·47
	·80
Caligny	·43
Foex	·55
Dartige's balance ...	·72
Belidor	not used.
Huelgoat	·45
Pfetsch	·771

	Proportion of Motive Power yielded.	Proportion of Water raised.
FIRE ENGINES.		
Merryweather	·572	·920
Tylor	·625	·887
Letestu	·452	·910
Perry	·300	·910
Flaud	·194	·920
Perrin	·210	·900
DRAINAGE PUMPS.		
Denizot	·690	·930
Delpech	·600	·926
Letestu	·513	·940
Millus	·502	
SUPPLY PUMPS.		
At Ivry (feeder alone) ...	·230	
At Ivry (three pumps)...	·530	
At St. Ouen	·696	
At Lisbon (Farcot) ...	·652	
Solid piston pumps ...	·900	
OSCILLATING PUMPS.		
Vascile's fire-engine ...	·50	
Gray's oscillating ...	·45	

(From Neville.)

Overshot wheels ...	·76
Breast wheels ...	·52
Very wide breast-wheels	·70
Undershot wheels...	·33
Floating turbine ...	·38
Impact turbine	·16
	·40

Barker's mill ...	·16
	·35
Ballysillan vortex ...	·75
Tremont vortex	·794
Montgolfier's ram ...	·65
Easton and Amos ram ...	·67
Water-pressure engines...	83

MISCELLANEOUS TABLES—*continued.*

Duty of Indian Hydraulic Contrivances.

Machine.	Details.				Work Done.			Power exerted.	Co-efficient of reduction.	Power utilised.
	Lift in feet.	Men.	Oxen.	No. of lifts per hour.	Cubic feet raised.		Gallons per day of 8 hrs.			
					per lift.	per hour.		foot-pounds per second.		foot-pounds per second.
Southern India.										
1. Picotah	10	2	...	600	·83	498	24,900	80	·80	64
2. Picotah	20·5	2	...	300	1·44	432	21,600	141	·80	112
3. Picotah	20·5	2	...	240	1·60	384	19,200	127	·80	101
4. Dal	10	6	...	1320	·33	443	22,150	71	·70	50
5. Mot	11	...	2	44	1·77	78	3,900	14	·70	10
6. Mot	20	...	2	90	2·00	180	9,000	58	·70	40
7. Mot	45	...	2	32	1·75	56	2,800	40	·70	28
8. Double mot	22·5	...	2	180	1·32	238	11,900	87	·60	52
					per minute.					
9. Common pump	10	1	...	60	7·77	466	23,300	74	·50	37
Barar.					per lift.					
10. Mot	16	1	2	35	5·81	203	10,150	52	·70	36
Northern India.										
11. Baling	5	2	...	1200	·33	400	20,000	32	·75	24
12. Beam and bucket	16	1	...	180	·45	82	4,100	20	·80	16
13. Windlass	4	1	...	84	2·00	168	8,400	11	·50	5
14. Dal	5	2	...	84	5·00	420	21,000	33	·70	23
15. Single mot	40	1	2	70	3·00	560	28,000	132	·70	92
16. Double mot	40	1	2	85	3·00	255	12,750	164	·60	98
17. Single chain of pots	40	1	2	168	·75	126	6,300	160	·55	88
18. Double chain of pots	40	1	2	138	2·40	330	16,500	212	·60	127
19. Common pump	25	4	...	2400	·10	240	12,200	97	·50	48
20. Lift and force pump	40	4	...	1440	·20	288	14,400	184	·60	110

MISCELLANEOUS DATA—*continued.*

Constants of Labour. (Hurst.)

EARTHWORK.

Excavator's Work per cubic yard, in terms of a day's labor of 10 hours, for different descriptions of soil.

Days of a Laborer.	Materials.		
	Soft. Days.	Moderate. Days.	Hard. Days.
Excavating only	·050	·100	·200
" in rock requiring blasting ...			·450
	Light.	Heavy.	Wet mud.
Throwing 5 feet high, or filling trucks ...	·048	·055	·065
Filling barrows	·045	·052	·061
Removing with wheelbarrow to 25 yards' distance	·026	·030	·030
Filling at backs of walls	·048	·055	·058
Ramming earth in 6-inch layers	·040		
" " 12-inch	·025		
Levelling earth from barrow-heaps without throwing	·012	·019	

Levelling and trimming slopes per s. yard	·020 to ·030
Turf 4 inches thick, cutting and stacking only, per s. yard	·045
" " resodding only, per s. yard ...	·065

Days of driver, horse, and cart.	
Removing 220 yards distance, per c. yard	·035 to ·040
Each additional 220 yards " "	·020 to ·025

N.B.—The vertical transport of earth is equal to 15 times the same horizontal distance when barrows are used, and 12 times when horses and carts are employed.

Days of an Indian Coolie.	Sand.	Gravel.	Stony Soil.
Excavating down to 9 feet, carrying to 25 yds. in a basket and depositing up to 6 ft. ...	1·25	2.00	3·75
Excavating down to 15 feet " "	2·00	2·75	4·50
Add for each 3 feet more of depth or height of delivery, or for each 15 yards' additional distance...	·25	·25	·25

MISCELLANEOUS DATA—*continued.*

Constants of Labour. (Hurst.)

BRICKLAYERS' WORK.

The time in days of 10 hours in which work can be performed.

One Bricklayer's Laborer.		Days.
Mixing concrete, wheeling and throwing from a stage,	1 c. yd. in	·300
Mixing mortar with a shovel	1 c. yd. in	·720
A two-horse pug-mill mixes 25 cubic yards of mortar ...	in	1·
Picking up and stacking bricks without moving, per 1,000	...	·150
" " if handed to him	"	·100
Selecting bricks for facings	...	·300
Taking down old brickwork in mortar, cleaning and stacking	1 c. yd. in	·410

One Bricklayer and Laborer.		Days.
Brickwork in mortar to walls, exclusive of face work,	1 c. yd. in	·320
" in cement	"	·373
" in mortar to covering arches	"	·410
Pointing flat joint in mortar and raking out mortar joints	1 s. yd. in	·110
Pointing flat joint in cement and raking out cement joints	1 s. yd. in	·170
Pointing tuck in cement and raking out cement joints	"	·258
Paving with stock bricks on edge in mortar ...	"	·086
" " " in cement ...	"	·100
Laying and jointing in cement 3-inch drain pipes	1 l. yd. in	·024
" " " 6 "	"	·048
" " " 9 "	"	·069
" " " 12 "	"	·093
" " " 18 "	"	·150

One Bricklayer only.		Days.
Working each fair face to brickwork and pointing	per s. yd.	·080
Working each fair face in malms or facing of superior bricks	per s. yd.	·117
Working each fair face in malms, circular to template	per s. yd.	·189
Rough cutting to brickwork	"	·135
Fair " "	"	·360

MISCELLANEOUS DATA—*continued.*

Constants of Labour—(Hurst.)

Masons' Work.

Days of a Laborer.			Days.
Rubble Stone.—Filling barrows	per cubic yard		·060
„ Removing 25 yards, and returning	„	„	·040
„ Unloading barrows	„	„	·030
„ Taking down old masonry in mortar, cleaning and stacking	„	„	·600
Breaking stone to 1¼″ ordinary limestone ...	„	„	·700
Do. do. granite or very hard stone ...	„	„	·930
Spreading the same for metalling 3″ deep ...	per square yard		·022

Days of a Mason and Laborer.			Days.
Rubble masonry, dry in foundations	per cubic yard		·240
„ „ in mortar above foundations...	„	„	·310
„ „ all beds being horizontal ...	„	„	·480
„ „ in cement do.	„	„	·570
Ashlar masonry, 12″ thick and in 12″ courses, rubble with chisel-drafted margins	„	„	2·160
Cubed stone hoisted and set in mortar	„	„	·756
„ „ „ in cement	„	„	·945

Days of a Mason only.			Days.
Add to rubble masonry for each fair face ...	per square yard		·090
„ „ if hammer dressed ...	„	„	·360
„ „ if curved	„	„	·414
Squaring 2″ flags for paving	„	„	·072
„ 4″ „	„	„	·135

Days of a Mason on various sorts of stone.

			Caen. Days.	Portland. Days.	Granite. Days.
Whole sawing, or axing,	per	square yard	·270	·540	1·270
Plain work	„	„	·540	·765	1·800
„ circular	„	„	·900	1·395	2·160
Sunk work	„	„	·675	1·080	2·135
„ circular	„	„	1·035	1·575	2·925
Moulded work	„	„	1·395	1·800	3·825
„ circular	„	„	1·800	2·700	4·905

MISCELLANEOUS TABLES—*continued.*

Cartage Table. (Hart.)

Constants of Labor per ton and per 100 cubic feet in terms of a day's work of a cart.

Distance of "lead" in miles.	No. of trips.	Cost of one trip.	Constants for one ton. For a weight of load of				Constants for 100 cubic feet. For a capacity of load in cubic feet of								
			8 cwt.	8½ cwt.	9 cwt.	10 cwt.	6	7	8	9	10	11	12	15	16
3/16 to ¼	16	·0625	·156	·149	·139	·125	1·042	·893	·781	·694	·625	·568	·521	·417	·391
¼ „ ⅜	12	·083	·208	·196	·185	·167	1·389	1·190	1·042	·926	·833	·757	·694	·556	·521
⅜ „ ½	10	·1	·25	·235	·222	·2	1·667	1·429	1·25	1·111	1·	·909	·833	·667	·625
½ „ ¾	8	·125	·313	·294	·278	·25	2·083	1·786	1·563	1·389	1·25	1·136	1·042	·833	·781
¾ „ 1	6	·167	·417	·392	·370	·333	2·778	2·381	2·083	1·852	1·667	1·515	1·389	1·111	1·042
1 „ 1¼	5	·2	·5	·471	·445	·4	3·333	2·857	2·5	2·222	2·	1·818	1·667	1·333	1·25
1¼ „ 1¾	4	·25	·625	·588	·556	·5	4·167	3·571	3·125	2·778	2·5	2·273	2·083	1·667	1·563
1¾ „ 2½	3	·333	·833	·784	·741	·667	5·556	4·762	4·167	3·704	3·333	2·030	2·778	2·222	2·083
2½ „ 3½	2	·5	1·25	1·176	1·111	1·0	8·333	7·143	6·250	5·556	5·	4·545	4·167	3·333	3·125
3¼ „ 4½	1½	·667	1·67	1·57	1·48	1·33	11·111	9·524	8·333	7·407	6·667	6·061	5·556	4·444	4·167
4½ „ 5¼	1¼	·8	2·0	1·88	1·78	1·6	13·333	11·429	10·	8·889	8·	7·273	6·667	5·333	5·
5¼ „ 8	1	·1	2·5	2·35	2·22	2·0	16·667	14·286	12·5	11·111	10·	9·091	8·333	6·667	6·25

MISCELLANEOUS DATA—*continued.*

Indian Coinage, Weights, and Measures.

The reductions from Indian data in the statistics are based on the assumption that the Rupi is equivalent to two shillings, and the Man or Maund to 80 lbs. avoirdupois. To aid the reader in any reductions from casual Indian data he may wish to make, the following equivalents may be useful.

The Rupee is the basis of British-Indian coinage and weights, and its weight is called a Tola.

				£.	s.	d.		Grains Troy.
1 Pie			=			$\frac{1}{8}$	and weighs	$33\frac{1}{3}$
1 Anna	=	12 Pie	=			$1\frac{1}{8}$	and weighs	400
1 Rupee	=	16 Annas	=		2	0	and weighs	180
1 Mohar	=	16 Rupees	=	1	12	0	and weighs	180

The established British-Indian weights are:—

1 Tola	=		=	·41143 oz. Avoir.
5 Tolas	=	1 Chittak	=	2·05714 oz. Avoir.
16 Chittaks	=	1 Seer or Ser	=	2·05714 lb. Avoir.
40 Seers	=	1 Man or Maund	=	82·2857 lb. Avoir.

The Seer is nearly a Kilogramme.

1 lb. Troy weighs 32 Tolas, and 1 lb. Avoirdupois 38·89 Tolas.

There are no measures of capacity, liquid and dry goods being estimated by weight.

The measures of length are the English yard or gaz, and the English mile, which has now superseded the very variable kos.

The measure of surface, the bigha, is not yet generally superseded by the English acre, its value in different places is:—

In Bengal ...	1600 s. yards.
At Banaras and Ghazipur	3136 s. yards.
The Madras Kani	6400 s. yards.
At Bombay ...	3406 s. yards.
In Northern India	3025 s. yards.
In Orissa ...	4840 s. yards.
In Tirhut ...	4225 s. yards.
The English acre .	4840 s. yards.

The local weights, the seer, man, and kandi, vary everywhere in Southern India; in the towns of Madras and Bombay they are thus:—

Madras.

1 Seer	=	$\frac{1}{8}$ viss	=	10 oz. Av.
40 Seers	=	1 Man	=	25 lb. Av.
20 Mans	=	1 Kandi	=	500 lb. Av.

Bombay.

1 Seer			=	11·2 oz. Av.
40 Seers	=	1 Man	=	28 lb. Av.
20 Mans	=	1 Kandi	=	560 lb. Av.

The other local weights and measures are both voluminous and doubtful, varying in almost every district.

HYDRAULIC MANUAL.

PART II.

CONSISTING OF

HYDRAULIC STATISTICS

AND

INDIAN METEOROLOGICAL STATISTICS,

FOR THE USE OF ENGINEERS.

COLLECTED AND REDUCED

BY

LOWIS D'A. JACKSON, A.I.C.E.

LONDON:
W. H. ALLEN & CO., 13, WATERLOO PLACE, S.W.
1875.

HYDRAULIC STATISTICS.

The Dynamic Force of Gravity at the Sea Level, and the Mean Temperature, for various Latitudes.

	Gravity.	Latitude.			Temperature.
		°	′	″	
Spitzbergen	32·2526	79	49	58	...
Greenland	32·2435	74	32	19	...
Uleo	...	65	30	0	34·38
Unst (Shetlands) ...	32·2173	60	45	25	...
Abo	...	60	27	0	40·28
Leith	32·2040	55	58	41	...
Edinburgh	32·2040	55	57	0	45·64
London	32·1908	51	31	8	50·74
Dunkirk	32·1895	51	2	10	...
Paris	32·1820	48	50	14	53·65
Bordeaux	32·1691	44	50	26	57·82
Montpelier	...	43	36	0	59·03
Toulon	32·1668	43	7	9	...
New York	32·1600	40	42	43	...
Mississipi	32·1380	35	0	0	...
Cape of Good Hope ...	32·1403	33	55	15	...
Sydney	32·1412	33	51	39	61·3
Rio de Janeiro	32·1121	22	55	13	...
Chandarnagar	...	22	52	0	75·10
Mauritius	32·1147	20	9	19	...
Bombay	...	18	53	0	80·60
Jamaica	32·1050	17	56	7	...
Trinidad	32·0917	10	38	56	...
Sierra Leone	32·0927	8	29	28	...
Ascension	32·0959	7	55	48	...
Colombo	...	6	58	0	80·90
St. Thomas Isle	32·0930	0	24	41	...
Rawak Isle	...	0	1	34	...
Equator	...		...		81·50
Gravity used in the working tables	32·2				

Statistics (mostly from Beardmore) for a few Rivers in various parts of the World, given for comparison with Indian Rivers.

	N	Q_2	Q_1	q_2	q_1	n_2	Proportion to down-fall. (1.)
	Catchment Area	Mean annual discharge.	Maximum or minimum discharge.	Mean discharge per square mile.	Maximum or minimum discharge per square mile.	Depth run off annually.	
	Sq miles.	C. ft. per sec.	C. ft. per sec.	C. ft. per sec	C. ft. per sec.	Feet.	
Mississipi	886 000	550 000	1 200 000	·62	1·13	0·70	·24
Ganges at Benares	180 000	250 000	1 285 000	1·38	7·15	1·57	
" at Kot	192 000	...	(min.)13 800	...	(min.) ·071	...	
" at Sikrigali	330 000	500 000	1 800 000	1·51	5·45	1·71	
Nile at Cairo	600 000	167 000	362 000	·28	0·60	0·32	
Kaveri	32 000	17 000	320 000	·53	2·85	0·60	·75
Po at Pontelagoscuro	26 754	61 000	221 000	2·27	8·15	2·57	
Ticino below L. Maggiore ...	2 420	11 000	63 000	4·45	8·15	5·03	
Adda below L. Como	1 670	7 000	28 000	3·95	16·91	4·47	
Rhine at Lauterburg	63 000	39 000	177 000	·63	2·49	0·71	
Rhone at Avignon	35 745	61 000	353 000	1·69	9·71	1·91	·75
Garonne at Armande	21 028	24 000	371 000	1·10	18·51	1·35	·66
Seine at Paris	17 111	8 000	63 000	·51	3·71	·57	·36
Thames at Staines	3 086	1 600	6 600	·54	2·16	·61	
Medway at Preston	481	...	2 300	...	4·85	...	
Shannon at Killaloe	4 571	...	73 000	...	16·00	...	

N.B.—**$n_2 = 1·13 \times q_2$ in the above columns, for feet of depth run off annually.**

Physical characteristics of a few Rivers in various parts of the world (from Stevenson) given for comparison with Indian Rivers.

	Length.	Drainage area.	Ordinary discharge.	Tidal portion.	Surface slopes on the lower portions only.	
	Miles.	Sq. miles.	C. ft. per sec.	Miles.	Feet per mile.	
Mississipi and Missouri ...	4400	1 226 600	665 900	...	O. ·233 F. ·271	From Ohio to the sea.
Amazon	4000	...		400	·195	
Nile	2240	520 200	23 100	...	L. ·271 H. ·458	Cairo to the sea.
Ganges	1680	432 480	207 000	...	·281	Rajmahal to Mirzapur.
Irrawadi	...	...	75 000	105	L. ·133 H. ·317	
Rhine	700	88 853	66 000	...	·642	Cologne to the sea.
Rhone	560	38 329		...	2·015	Besançon to the sea.
Thames	204	5 000	1 700	66	·076	Teddington to Yantlet Creek.
Severn	180 Nav	8 580	..	68	·583	Inward point to Austhead.
Clyde	98	945	800	22	·108	Broomielaw to the sea.
Boyne	60	700	3 000	2		

N.B.—O. means ordinary; F. flood; L. low-water; H. high-water.

Details of the Breadths, &c. of various large Rivers at their Entrances (from Heywood).

	Ganges.	Kistna.	Godaveri.	Kaveri.	Mahanaddi.	Mississipi.
Extreme breadth	5¼ miles	1⅓ miles	4 miles	1 mile	3 miles	...
Extreme breadth of channel	2¼ „	1¼ „	2½ „	¾ „	...	4000 feet
Least breadth of channel	1¼ „	¾ „	1⅓ „	⅓ „	⅓ mile	2750 „
Fall per mile, in feet	...	1·17	1·09	3·5	1·67	0·27
Rise in mansun, in feet	27	35	30	12	32	51
Greatest depth in dry season	30	10	10	6	...	75
Surface current in floods	4 to 7 miles	7⅓ miles	4¼ miles	6 miles	...	5 miles
Flood section, in square feet	288 000	153 000	216 000	37 800	...	240 000
Flood discharge, in cubic feet per second ...	1 800 000	1 500 000	1 500 000	300 000	1 800 000	1 500 000
Least discharge in dry season	45 000	1 125	2 250	None	...	...
Longest duration of flood	40 days	10 days	10 days	10 days	12 hours	80 days
Area of delta, in square miles	...	...	3000	...	5000	12 300

The Areas of the River Basins of India.

Western Basins.

	Square miles.
Indus	372 700
Thar Desert	68 700
Luni	22 400
Katiwar and Kach peninsulas	27 600
Mahi	15 500
Narbada	36 400
Tapti	27 000
Western Ghats and Coast series	41 700
Sabarmatti	9 500
Western Banas ...	6 300
Dhardur	1 800
Total ...	629 600

Burmese Basins.

Irawadi	158 000
Sittang...	18 300
Salwen	62 700
Arakan Basins ...	29 700
Tenasserim Coast ...	14 200
Total ...	375 700
Brahmaputra	361 200

Eastern Basins.

	Square miles.
Ganges	391 100
Sabanrekha	11 300
Baitarani	11 900
Brahmani	15 400
Mahanaddi	43 800
Godavari	112 200
Orissa Coast	22 200
Coromandel Coast ...	10 300
Lake Pulicat	6 700
Lake Koler	3 100
Kistna...	94 500
Pennar	20 500
Palar	6 300
South Pennar... ...	6 200
Vellar	4 500
Kaveri...	27 700
Vaiga	9 800
Tambaravari	3 600
Vaipar...	3 900
Total ...	705 000

Lateral curves of rivers of fixed regimen. (*Fergusson.*)

River Lengths.	Direct distance	Distance by river.	Width of stream, low water, dry season.	Oscillations.	Length of oscillation.
	Miles.	Miles.	Feet.	Number.	Miles.
Ganges.					
Allahabad to Chunar	62	104	3500	17	3·7
Chunar to Baxar	80	113	4000	20	4
Baxar to Patna	74	96	5000	15	5
Patna to Manghir	82	106	6000	11½	7
Manghir to Rajmahal	96	108	7000	10	9·5
Rajmahal to Rajapor	90	100	7000	10	9
Rajapor to Patna	30	44	4000	6	5
Patna to Jafirganj	32	36	3000	8	4
Bhagaratti:					
Choka to Naddia	96	120	1200	62	1·5
Naddia to Chogdah	24	30	2000	9	2·5
Chogdah to Calcutta	34	42	3000	11	3
Jellinghi.					
Jellinghi to Naddia	50	112	1000	42	1·2
Mata bagh.					
Ganges to Cumar	18	28	1500	9	2·0
Cumar to Kissanganj	30	50	800	46	·66
Kissanganj to Chogdah	24	29	500	47	·5

Fall in feet per mile of Indian Rivers.

Northern India.

The Ganges.	
at Sukertal . .	1·5
from Gurmaktesar to 60 miles south .	1·25
Khanpur to Allahabad .	·75
The Bhagiratti	
for 190 miles; between Rajmahal and Mirzapur . . .	·281
The Jamna,	
at Agra . . .	1·25
The Indus,	
near Sakkar . . .	·75
The Son, in the Punjab,	
at Lahor-road bridge	14·
The Markanda,	
at Hassanpur . .	2·72
The Mahanaddi,	
lower 100 miles .	1·4
next 100 miles . .	2·2

Southern India.

The Godavari,	
Sironcha to Palmilla .	·5
Enchampilli to Damagudiam . .	1·0
Damagudiam to head of delta . .	·5
through delta to sea .	·5
The Pranhita,	
Tallodhi to Sironcha, 90 miles . .	1·0
The Warda,	
above the Wunna .	4·0
below it to Tallodhi .	1·0
The Wainganga,	
Kampti to Tallodhi, 192 miles . .	2·8
The Kistna,	
Bezwara to sea . .	1·0

Southern India.

The Tambrapurni,	
at Strivigantam	2·5 to 3·0
The Tungabaddra, Dharwar . . .	2 to 2·5
The Warda, Dharwar . .	2·
The Malparba, Belgaum	1·25 to 1·5
The Gatparba, Belgaum,	
below Gokak .	1· to 2·
The Nira, Puna,	
above Ramlishwar .	4·6
The Indarauni, Puna .	2·75
The Bhima, Puna,	
Sarwali to Deksal .	2·75
The Siena, Sholapur,	
above Undogaum .	2·75
The Krishna, Sattara,	
above Kursi . .	4·7
Kursi to Bahey . .	1·9
Bahey to Yerla . .	1·4
below Yerla . .	·6
The Koina,	
Helwak to Karrar .	1·3
Karrar to Bahey .	·4
above Bamnoli . .	6·0
The Yerla,	
Krishna to Chickli .	8·8
The Mann,	
Diguchi to Manswar .	5·5
The Kaveri,	
above the Kalerun .	3·5
thence to Seringham	3·5 to 2·0
Seringham to sea .	1·0
The Kalerun,	
from the Kaveri to Seringham .	3·1 to 1·6
Seringham to sea	1·6 to ·6

Flood discharges of Indian Rivers, according to various reports.

	Catchment Area.	Flood Discharge.	Discharge per sq. mile.	Coefficient (n) in the formula.
	Sq. miles.	C. ft. per sec.	C. ft. per sec.	*
NORTHERN INDIA.				
Ganges at Rajmahal	286 000	1 350 000	4·7	1·1
Combined Mahanaddi and Katjuri in flood of 1834 ...	67 000	1 850 000	27·6	4·6
Jamna at Allahabad	118 000	1 333 000	11·3	2·
Son (Bengal) at causeway ...	34 000	1 700 000	50·0	7·
Indus at Sakkar	250 000	380 000	15·2	0·3
Son (Punjab) at Lahor-road bridge	3 600	96 000	26·6	2·
Markanda at Hassanpur, 1845 .	1 200	47 838	39·8	2·
Sai at Rai Bareli bridge ...	960	16 500	17·2	1·
Sai at railway bridge	240	12 000	50·0	2·
Gumti at Lakhnau bridge ...	2 000	22 366	11·2	0·8
Gumti at Saltanpur bridge ...	3 600	39 000	10·8	0·8
Loni at railway bridge ...	120	4 600	38·3	1·3
Kalliani at Lakhnau bridge ...	360	17 758	49·3	2·1
Morna (Berar) at railway bridge	211	122 715	581·	20
Nalganga at railway bridge ...	213	153 846	722·	24
SOUTHERN INDIA.				
Godavari at Rajamandri ...	120 000	1 350 000	11·2	2·3
Kistna at Bezwara	110 000	1 188 000	10·8	1·9
Tumbadra at Karnul	20 000	270 000	13·5	1·6
Kaveri at Frazerpett	415	111 000	267·3	12·5
Kaveri at Seringham	28 000	472 500	16·9	2
Penner at Nellor	20 000	359 100	18·1	2
Palar at Arcat	3 700	270 000	74·2	5·7
Tambrapurni at Palamcotta ...	587	189 000	324·0	16
Chettar at Alligyapandrapuram	486	29 700	60·8	3
Vigay at Madura	1 600	43 200	27·0	2
Manjilanthi at Balagunta ...	90	10 800	121·5	4
Gadanamathi	29	28 088	972·0	23·
Varhazanamathi at Periacolam	41	8 100	202·5	5·
Irriti (Malabar)	336	149 850	446·0	19·

* See pages ix. and lxx. of working tables

Flood Discharges of Indian Rivers on the East-Indian Railway, by S. Power, Esq., Chief Engineer.

	Waterway.			Recorded flood level.	Estimated rainfall run off through, in inches per hour.	Discharge per square foot of waterway to carry off estimated rainfall.	Mean velocity through in order to carry this off.	Estimated addition to waterway necessary to reduce this velocity below 2·5, existing waterway being = 1.	Rainfall of district.
	Catchment area in square miles.	Lineal feet.	Square feet below recorded flood level.			C. ft. p. sec.	Feet p. sec.		
Kurriamnassa	3 400	4 000	39 000	11	·25	162	15	6	Comparatively light.
Son	23 000	14 200	172 500	8	·125	11	10	4	
Punpun and Hallshar valleys...	9 000	11 086	123 648	14	·25	162	11	4	
Kinal River	1 100	1 542	11 002	10	·25	162	16	6	
Hill streams west of Jamalpur	240	1 670	7 385	80	·5	323	10	4	
Do. Jamalpur to Sahibganj	2 650	6 796	59 486	23	·5	323	14	5	
Do. Sahibganj to Tinpahar	52	1 641	7 952	155	.5	323	2	1	Heavier than that west of Manghir.
Do. Tinpahar to Baluva ...	66	1 176	2 476	144	·5	323	2	1	
Gumani River	520	1 639	10 165	20	·5	323	16	6	
Hill streams between Gumani and Mullarpur	1 200	5 824	53 301	44	·5	323	7	3	
Adjai and Mor valleys ...	3 640	7 752	120 300	30	·5	323	10	4	
Khanu to Haurah	...	14 000	65 000	...	...	...	...	...	

Discharges of Indian Rivers.

The River Son, Bengal.

Date.	Place.	Discharges in cubic feet per second.
8 Jan. 1855	At causeway and headworks of canal.	5 750
1 Feb. 1855	„	4 624
1 Mar. 1855	„	11 020
Ordinary minimum	„	4 000
Extreme drought	„	960

The Mahanaddi and Katjuri.

Flood of 1855	Below junction with the Beropa	1 040 000
Flood of 1855	The Katjuri and Kokai.	780 000
	Mahanaddi Series, Total	1 820 000

The River Jamna.

6 June 1872	Mandawala	1 388
6 June 1872	Bud	5 126
29 July 1872	Chaogaon	144 890
19 Dec. 1872	Railway bridge	2 128
19 Dec. 1872	West Ghat	2 037
19 Jan. 1873	Railway bridge	2 554
20 Jan. 1873	West Ghat	2 934

The River Satlaj.

21 Jan. 1856	Proposed site for headworks of Canal.	2 781
12 Feb. 1857	„ „	4 135
26 Jan. 1859	„ „	4 027
20 Dec. 1859	„ „	4 663
21 Jan. 1861	„ „	4 461

N.B.—There is reason to believe that these are in excess.

Discharges of Indian Rivers—continued.

The River Ravi.

Date.	Place.	Discharges in cubic feet per second.
19 Dec. 1872	Shahdera, Lahor 94 miles.	703
19 Dec. 1872	Alpah, below escape 147 miles.	879
19 Dec. 1872	Bhátiah	509
19 Jan. 1873	Shahdera	687
19 Jan. 1873	Alpah	478
19 Jan. 1873	Bhatiah	271
19 April, 1872	Sidhuri	7 689
21 Sept. 1872	,,	13 452
19 Dec. 1872	,,	1 866
20 Jan. 1873	,,	2 296
19 Mar. 1873	,,	3 579

The River Bias.—At Naushehra.

19 April, 1872	7 498	19 Dec. 1872.	4 901
19 Oct. 1872	8 797	19 Jan. 1873.	5 117
19 Mar. 1872	3 464 at Pakhowal		

The River Indus.—At Kalabagh.

In Dec. 1871	21 220	Jan. 1873.	20 541
Jan. 1872	18 657		
Dec. 1872	21 878		
Jan. 1873	20 781		
Jan. 1873	18 657 at Dera-Ghazi-Khan.		

The River Kuram.—At Kalabagh.

In Jan. 1873. 545 (included with the Indus discharges).

The River Indus, in 1872-73.—At Dera-Ghazi-Khan.

Average gauge readings for each month.

April, 1872	6·27	Aug. 1872	7·97	Dec. 1872	3·46
May, 1872	7·32	Sept. 1872	6·19	Jan. 1873	3·55
June, 1872	9·28	Oct. 1872	4·85	Feb. 1873	3·23
July, 1872	9·81	Nov. 1872	3·98	Mar. 1873	3·52

Rivers of Maisur.—The Kistna System.

The Kistna River System.	Feeders in Maisur.	Length in Maisur.	Area over which the drainage is unintercepted in tanks in Maisur.	Area over which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	Percentage of whole area under tank system.	Rises	Falls into
		Miles.	Sq. miles.	Sq. miles.	Sq. miles.	per cent.		
Kistna ... (Falls into Bay of Bengal in lat. 15° 45′.)	Warda	47	430	180	610	·29	Sagar, has a few minor anicuts.	River Tungabaddra, 35 miles below Harrihar; may be further utilized.
	Choardi	43	None.	510	510	100·00	NE. of Kaulidrug, has a few minor anicuts.	River Tungabaddra, 10 miles above Harrihar; may be further utilized.
	Tunga	149	1287	100	1 389	7·25	Gangamula, lat. 13° 15′. Many anicuts on feeders.	Kudli, 10 miles NE. of Shemogah.
	Baddra	160	1500	175	1 675	10·50	Do. Many anicuts on feeders.	Do. Proposed Lackawally reservoir.
	Sulikerri ...	45	231	799	1 030	77·60	S. of Chennagherri, feeds the Sulikerri tank.	River Tungabaddra at Harrihar.
	Chinna Haggri ...	53	168	356	524	67·90	(Not given.) Might be utilized; feeds the Haggri.	NE. of Mulkalmora, Ballari.
	Haggri or Vedavatti Yerahalli	114	1198	4097	5 295	77·37	Bababudin Hills, lat. 12° 30′; feeds Eyenkaira and Maddak tank, also the Mauri Kunwai site; should be further utilized.	River Tungabaddra, 55 miles above Sunkesala.
Total of the Kistna System.	...	611	4814	6217	11 031	56·17		

The Rivers of Maisur.—The Pennar System.

The Pennar River System.	Feeders in Maisur.	Length in Maisur.	Area over which the drainage is unintercepted in tanks in Maisur.	Area over which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	Percentage of whole area under tank system.	Rises at	Falls into
		Miles.	Sq. miles.	Sq. miles.	Sq. miles.	per cent.		
Palar ... (Falls into the Bay of Bengal in lat. 12°27'.)	Palar	47	None.	1036	1036	100·00	Chintamânipett Colar; this is entirely utilized by tanks in Maisur.	Enters Madras territory at Gumsur.
Penner ... (Falls into the Bay of Bengal in lat. 14°37'.)	Gandacholli or Jimangal ...	60	185	452	637	77·07	Davroydrug, Tomkur; not used now, might be utilized.	
	Upper Penner ...	36	149	501	650	...	North of Nandidrug; not used, might be utilized.	
	Kushawatti ...	16	None.	993	993	100·00	All rise in north of Colar Division; feed Darmavaram tank, the Kuchru tank, and Goodybanda large tank.	
	Chittravatti ...	23						
	Papakenni ...	32						
Total of the Pennar system ..	...	167	334	1946	2280	85·35		
Pennar ... (Falls into the Bay of Bengal in lat. 11°25'.)	Verushavatti ...	18	135	259	394	65·75	In Colar; not likely to be further utilized.	Joins the Penankenni.
	Penankenni ...	14	87	1060	1147	92·41	NE. of Nandidrug; feeds five large tanks, would not be further utilized.	Passes Urcottah.
Total of Pennar system ...	...	32	222	1319	1541	85·60		

The Rivers of Maisur.—The Kaveri System.

The Kaveri River System.	Feeders in Maisur.	Length in Maisur.	Area over which the drainage is unintercepted in tanks in Maisur.	Area over which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	Percentage of whole area under tank system.	Rises at	Falls into
		Miles.	Sq. miles.	Sq. miles.	Sq. miles.	per cent.		
Kaveri ... (Falls into the Bay of Bengal in lat. 11°55'.)	Upper Kaveri ...	171	1201	750	1 951	38·44	Tallakaveri, lat. 12° 25'; has large anicuts on it, which require improvement.	Passes Talkad.
	Surnavatti ...	23	185	None.	185			
	Hemavatti ...	107	630	662	1 292	51·25	E. of Bellariadrug, lat. 13° 10'; has large anicuts on it, no tanks.	
	Yegachi	37	375	145	520	27·90	S. of Bababudin hills; feeds Chikmanglur tank, and some anicuts.	Joins the Hemavatti.
	Lachmantirth ...	64	487	175	662	26·44	Brammeagherri, lat. 11° 55', no tanks, many anicuts.	Joins the Kaveri.
	Lokani	27	80	95	175	54·30	Supplies a large tank, the Motit also.	Northern feeder of the Kaveri.
	Kabbani ...	80	843	784	1 627	48·18	Lat. 11° 55'; its feeders supply large tanks and anicuts, to be further used.	Kaveri, 12 miles above Talkad.
	Shimsha ...	73	585	2639	3 224	81·85	From Gandestri to Tomkur, large tanks and anicuts, but might be further utilized.	Kaveri at Sivasamudram.
	Arkavatti ...	63	1140	519	1 659	31·30	Nandidrug; five tanks and some ruined ancuts.	
Total of Kaveri system	...	646	5526	5769	11 295	51·75		

The Rivers of Maisur.—The Western Coast System.

The Western Coast River system.	Feeders in Maisur.	Length in Maisur.	Area over which the drainage is unintercepted in tanks in Maisur.	Area over which the drainage is intercepted by tanks in Maisur.	Total area of catchment basin.	Percentage of whole area under tank system.	Rises at	Falls into
		Miles.	Sq. miles.	Sq. miles.	Sq. miles.	per cent.		
Western Coast Rivers.	Garsappa or Sheravatti ...	49	1 101	*None.	1 101	...	All rise to the west of the Ghats, are useless to Maisur, except the Sheravatti, on which are a few channels.	
	Nartravatti ...	6	780	None.	780	...		
	Puiswanni ...	12						
	Komardari ...	16						
	Other names not given	20						
Total of Western Coast System	...	103	1 881	...	1 891			
Total for the rivers of Maisur and Curg...		1606	12 777	16 287	29 064	56·16		

Brief Accounts of Indian Rivers.

The *Indus* at Attock, certain recorded velocities are as follows,

In hot seasons, opposite fort, velocity 13 miles an hour.

At tunnel site, in cold season, 5 to 7 miles an hour.

Do. in hot season, 13 to 14 miles an hour.

Surface velocity at centre, Dec. 1869, 9 miles an hour.

The rise of ordinary floods is from 5 to 7 feet in 24 hours only, and is 50 feet above cold weather level. The flood of 1841 was 92 feet above cold weather level, and that of 1858, 80 feet.

Barra River, at the Lahor and Peshawar-road bridge, 7 miles west of Peshawar, the waterway allowed is 180 lineal feet. In the flood of July, 1861, the flood rose 18 feet in 5 minutes, and had a surface velocity of 15½ feet per second. The soil of the bed consists, first, of 18 feet of silt and loose sand, then 8 feet of firm sand resting on clay.

Son River, Panjab, at Lahor and Peshawar Road, has a catchment area of 573 sq. miles; maximum flood depth, 15′; mean velocity, 8 to 9 feet per second; slope of bed, 14′ per mile; calculated mean velocity, 13′ per second; flood discharge, calculated from sections, 91 000 cubic feet per second = ¼″ over the catchment basin; the perennial stream is never less than 1′ deep. Bed at surface boulders; at 11′, conglomerate blocks; at 16′, a hard, dry foundation; width of river at site 1000′, but a little above only 750; clear waterway of bridge, 945 lineal feet.

The Jamna.—At the Sirsawa bridge of the Delhi Railway, 37 miles SE. of Amballa, the waterway allowed is 2376 lineal feet; at this place the Jamna is constant, for six months, from April to September, being snow-fed; it rises in March and falls in October; at the site the soil is gravel and coarse sharp sand, above the bridge site it consists of large 14 lb. boulders. Its flood velocity is 8 miles an hour, scouring the bed, carrying along the boulders and depositing them 30 feet below the ordinary bed of the river. In 1867, the river rose in flood to two feet above its banks; in 1868, 14 inches above that again.

The floods of the Jamna at Allahabad were recorded by Mr. Sibley,

C.E., from 1861 to 1865, observations being taken daily at 6 A.M. and 6 P.M. The extreme variation of ordinary level within the five years' observations was 2 feet; the extreme variation of lowest level was generally also 2 feet. The lowest water occurred between the 19th and 28th April, when the rise from snow melting begins. The great rise due to the periodic rains generally begins on the 19th or 20th June. The highest flood generally occurred between 22nd and 26th of August; the highest flood recorded was in 1832, a little higher than that of 1861.

In 1861 R. L. high flood 161·6, 8 days over 155 and 4 days over 160.
1862 R. L. ... 144·5 lowest recorded flood.
1863 R. L. ... 155·
1864 R. L. ... 152·5

The floods of 1861 were exceptionally long in duration.—The lowest recorded flood was 30 feet above low water level, the average 40, and the maximum 50 feet: the maximum velocity measuring 950 feet in 81 seconds = 12 feet per second, and for 12 days being more than 10 feet per second. At the period of greatest discharge the mean surface velocity was 10 feet per second, and the mean sectional velocity 9 feet per second; the sectional area at that level being 145 000 square feet, the discharge per second was 1⅓ million cubic feet.

This river supplies the Eastern Jamna canal with about 1065 cubic feet per second, the Western Jamna canal with about 2500, and will also supply the Agra canal with 800 cubic feet per second.

River Markanda at Hassanpur, in 1859, by Mr. C. J. Campbell, C.E.

The bridge site, where the banks are well defined, is about three miles below Hassanpor.

Width of channel	1 577 feet.
Sectional area	6 938 square feet.
Hydraulic slope	2·72 feet per mile.
Mean velocity	5·15 feet per second.
Discharge	35 370 cubic feet per second.
Flood of 1845	47 838 cubic feet per second.
Flood depth	10 feet.
Ordinary flood depth	6 to 9 feet.
Waterway of bridge	1 073 lineal feet.
Height of roadway	24 feet above bed.
The soil of the bed is	Sand and silt for 40 feet in depth.

k

The *Son River*, in Bengal, is 425 miles long, rising near Ammar Kantak in Central India, the first 325 miles of its course are in rocky country; it emerges from the Kaimor hills at Rhotas, 100 miles from its confluence with the Ganges at Patna; the last 100 miles being in the plains. The river is three miles broad at Telothu; and generally in the plains is two miles in breadth; for eight months in the year the stream is a quarter of a mile broad. The extreme flood discharge is said to represent 2¾ inches of rainfall over the whole catchment area in 24 hours (the heavy floods never exceeding four days); in this state half the water is thrown over the country below Massaura. The lowest discharge in dry seasons is 4000 cubic feet per second. During the rainless year referred to in the table of discharges, the rain from June to October inclusive was at Shahabad, 21·3 inches; at Bahar, 18·9; and at Patna, 19·6; it is generally 35 inches at each; though in the year following the rainless year the fall at Patna was 50 inches.

At Dehri, a town 65 miles above Patna, are the headworks of the Son canals, and the causeway of the Grand Trunk road. The channel of the river here varies from 2 to 2½ miles in breadth, and has a fall of from 1·75 to 3 feet per mile, and its flood rise, or difference between summer and high-flood level, is from 14 to 20 feet; its discharge varies from 4000 to one million cubic feet per second. The bed is composed of shingly sand to a great depth.

It is unfortunate that the diagrams of the gaugings of this river, as well as those of the Ganges, the Kodra, the Kura, Punpun, Durganti, Chandarprobah, Kuramnassa, Morhar, and Sura, recorded by the engineers of the Son canals in 1872 and 1873, are not yet available.

The Ganges.—The discharges of this river given in the table, obtained from Beardmore's work, were taken under the following conditions:—

1st. The quantities at Benares were taken from a section by Prinsep, on the 25th April, 1829, after a long interval without rain: the area of the section was 48 650 square feet, the width 1400, the mean depth 34·75 feet, the mean velocity 23·5 feet per minute; the maximum discharge at the same place was computed, when the river was 3000 feet wide, and had an average depth of 58 feet, and sectional area 175 000 square feet, the mean velocity being about 440 feet per minute.

2nd. The gauging at Kot, near Balliah, was taken by Lieutenant Garforth, in the first week of May, 1850, when the river was at its lowest; the sectional area was 5876 square feet, width at water-level

1125 feet, mean velocity 141 feet per minute; the maximum velocity in mid-channel was 198 feet per minute, which greatly exceeded that in other places where the river was deeper; the maximum depth in this section was 9·42 feet in a narrow place only 120 feet wide, the remainder of the section varying from 4 to 6 feet in depth.

3rd. The gauging at Sikrigali was taken on the 9th March, 1829; at this place, 30 miles above the delta, the Ganges has received the Gogra, the Gandak, Kusi, Son, and other rivers, whose united volume is frequently more than that of the Ganges proper, Jamna, and other affluents which form the river at Banaras. The data for gauging were as follows: breadth about 5000 feet, depth 3 to 5 feet, sectional area 15 000 square feet, mean velocity about 86 feet per minute; in extreme freshes the breadth is about 10 000 feet, mean depth 28 feet, sectional area 280 000 square feet, the mean velocity being about 440, and the maximum 600 feet per minute.

The Ganges seems to have preserved its general course for ages down to Suti, 34 miles below Rajmahal, where, at some period within the range of tradition, some alteration in the banks caused it to be diverted from its former western course, now known as the Bhagiratti as far as Naddia and as the Hughli (not an indigenous name) below it, to its present eastern course by Rampur-Bauliah and Jellinghi which joins that of the Brahmaputra to form the Megna estuary.

There is a lamentable want of available accurate modern information as to the physical conditions and discharges of this river.

The Damuda.—This river rises in the Sonthal Hills, its upper portion and its tributaries being comparatively unknown; it becomes a single and defined channel at about 23 miles above Raniganj, and passing through the coalfields of that tract, enters the yellow clay of the delta near Burdwan, 52 miles below Raniganj, whence it continues to Selimabad. At Selimabad, 16 miles below Burdwan, is an old branch of the Damuda, which flows into the Hughli above the town of that name; but the present course is by Ompta to the Hughli, opposite Fulta, a length of 60 miles. This river is interesting on account of its floods frequently inundating the country; remedial measures, the improvement of its embankments and the damming up of the old branch, were unsuccessfully attempted in 1857 by various military engineers. There is a large amount of Governmental correspondence on this subject, but no valuable hydraulic data; in fact, the velocity tables of the floods give as a maximum 77 feet per second, or 5 miles an hour, or less than half what it must be. In 1872-73 some

hydraulic observations were made by the civil engineers employed on the Orissa canals, but the records are not yet available.

The Damuda, with a catchment basin of 7000 miles, has a flood discharge representing ·125 inch per hour of rainfall.

The Mahanaddi and its Tributaries.—Reduced levels of the flood and low water sections of the Mahanaddi for the last 200 miles.

At	Miles.	Flood. Feet.	Low-water. Feet.
Sonpor	0	365·5	335·5
Barmul Pass entrance	60	245·5	175 5·
Do exit	72	215·5	175·5
Kantilu	94	165.5	135·5
Baidessur	107	140·5	110·5
Chirchika	115	129·5	87·5
Naraj	135	92·5	65·5
Kattak	144	77·5	55·5
Mouth of Katjuri, Jaipor	172	37·5	15·5
Mouth of Mahanaddi	200	5·5	0
Mean Sea Level	...	...	0

The Tributaries of the Mahanaddi.

Torrents.	Near village of.	Distance above Kattak. Miles.	Width of Mouth. Feet.	Nature of bed.	Nature of stream.	Fall of bed per mile.
Kaligiri ...	Baidessur	37½	200	Alluvial.		
Komi	Kantilu	48¼	320	Rocky above		
Burtung...	Bentpara	64¾	300	Sandy and rocky.		6⅜
Salki	Above Boad	120½	465	Ditto.	Perennial.	
Baj	Dayah	136	700	Ditto and very rocky.	Perennial.	
Mirni	Lowpara	141	400	Sandy and rocky.		
Tel	Sonpor	143	3470	Ditto.	Perennial.	

The Mahanaddi and Katjuri have in high floods velocities of 7 feet per second. At Naraj the Mahanaddi emerges from a rocky ridge, only ¾ mile wide into a wide basin, 3 miles broad, and 4 miles long, reaching to Kattak. The head land of the delta at Naraj divides the Mahanaddi north of town from the Katjuri south of town. The

upper affluents of the Mahanaddi are in hilly country, and may be said to be unexplored.

From gaugings at Kattak it appears that the ordinary embanked channels of the delta could only carry off a flood rising to 20½ feet on the gauge, and half a flood rising to 27 feet—hence the devastation so often caused; a flood over 20½ feet may last seven days, although they remain at full height for only 12 hours. There is a sounding of 80 feet of water in the bed between Baidessur and Dewakot, being 16½ feet below mean sea-level. The Banki reservoir covers an area of 150 square miles, having a mean flood depth of 20 feet, and gives one-third of the relief from flood that is required. Total flood discharge from 27th July to 3rd August, 1855, 761 billion cubic feet; of which 545 billions can be carried off in the river channels, leaving 216 billions in 7 days = 400 000 cubic feet per second to be provided for by reservoirs, cuts, and special arrangements.

The historian of this river is Captain Harris, who laboured many years in endeavouring to mitigate the effects of its floods.

The Godavari rises at Nassik, lat. 20° 0′, long. 73° 47′, and passes south of Aurungabad, through native territory for 450 miles, until it joins the Pranhita at Sironcha. Its basin is about 60 000 square miles, or including its tributaries 120 000 square miles. Above Sironcha it is unnavigable, and had a discharge in February, 1866, of only 300 cubic feet per second. From Sironcha to Palmilla, about 38 miles, the fall of the bed is ·5 feet per mile, and this part of the river is navigable; the Pranhita having contributed a discharge of 726 cubic feet per second (Feb. 1866). From Palmilla to Enchampilli is a barrier of rock 14 miles long; known as the second barrier of the Godavari, above which the river is 1300 yards wide. From Enchampilli to Dammagudiam, 70 miles, the river has a fall of 1 foot per mile. At Dammagudiam there is a barrier of rock 8 miles long, known as the 1st barrier of the Godaveri; at this place the river is 1760 yards wide, the discharge being 1875 cubic feet per second in May, and 9375 cubic feet per second in January, having a current of 3 to 5 miles an hour. At Gollagudium, about 20 miles below this barrier, the discharge in Feb. 1866 was 2825 cubic feet per second. At Palaveram the river emerges from the hills, 80 miles below the 1st barrier, and 20 miles from the town of Rajahmandri, which is 4 miles from Dowlaishwaram, the head of the delta: for these 104 miles the fall is about ·5 feet per mile. At Palaveram the river gorge is only 200 yards wide (February, 1866), but the floods rise to 60 feet above the February level; very high freshes occur three times in the

mansun and last for four or five days; the general velocity of the stream then being 6 miles an hour. The river is navigable from Sironcha downwards, excepting at the barriers, during the mansuns only, *i.e.* from December to May. It has three unnavigable tributaries; the Indrawatti, joining it above the 2nd barrier, which is 300 miles long, discharging 150 cubic feet per second (Feb. 1866); the Sibberi, 200 miles long, discharging 500 cubic feet per second (Feb. 1866), and joining it below the 1st barrier; and the Jal, 100 miles long.

From Sironcha to the 1st barrier the river channel has no permanence of form, it shifts its course, and forms large banks and shifting shoals; the banks are soft, and the rocks that occur are sandstones and sometimes limestones. From the 1st barrier to the head of the delta the channel is comparatively permanent, the banks are tough, the sand is large and coarse grained, requiring a powerful current to displace it, the rocks are unstratified, and form natural groins, which aid in giving permanence to the channel. From the delta head downwards the river runs in a natural embankment, 6′ to 24 feet above the level of the country; its bed falls ·5 feet per mile, the summer water surface ·7 feet per mile, and the high flood surface 1·25 to 1·50 feet per mile, down to the mouth, 40 miles below. In the delta the river, when in full flood, has a width of 2¼ miles, and a surface velocity of 4½ miles an hour; the rise of surface varies from 20 to 50 feet; the last two feet of rise being never maintained for more than two hours. From the middle of June to the middle of September the volume is always more than 12 000 cubic feet per second; during the rest of the year 3000 cubic feet per second is considered its ordinary minimum supply. In excessively dry years the discharges have been as follows: December, 16 875 cubic feet per second; January, 8047; February, 3825; March, 2782; April, 2047; May, 1687; first half of June, 1500 cubic feet per second.

The Upper Tributaries of the Godavari, that together form the Pranhita, which is 90 miles long from Tallodhi to Sironcha, are the Warda 250 miles long, which rises in the Satpura range, and after being joined by the Wunna at the falls of Dindora, becomes navigable for the last 100 miles of its course; the Painganga, which rises in the hills south of Berar, and after an unnavigable course of 320 miles, joins the Warda above Chanda; and the Wainganga, which rises in the Satpura range north of Nagpur, takes a course of 430 miles, unnavigable, and joins the Warda at Tallodhi. The Pranhita is like the lower portion of the Warda navigable for three months in the year, from Tallodhi to Dewalmarri, where there is a barrier of rock

36 miles long; below this to Sironcha it is navigable for four months. The fall of its bed is about 1 foot per mile, so also is that of the Warda in its navigable portion. Above this the Warda falls 4 feet per mile, and the Wunna 2 feet per mile. The Wainganga has a fall of 546 feet in 192 miles, from Kampti to its mouth, or 2·8 feet per mile.

In 1864–67 an attempt was made by Col. Haig, aided by Captains Roberts and Jackson, to open a navigable communication from Dindora to the coast; it was, however, at last abandoned, on account of its excessive expense.

The Kistna rises north of Sattara, Bombay presidency, in latitude 18°, and enters the sea 35 miles SW. of Masulipatam; its catchment area being 30 000 square miles. It is a perennial river 600 miles long, entering the plains at 80 miles from its mouth, and there becoming an important river, is utilized in irrigation. In the dry weather, from November to June, its supply is very small, being derived principally from springs in its bed; from July to October it varies much, even falling as much as 10 feet in 24 hours. In full mansun there is a constant stream 20 feet deep, the crest of its banks is from 20 to 40 feet in height, and its section from 1¾ to 2¾ miles broad. At Bezwara, the head of the delta, 60 miles from the sea, where are the last outlying spurs of the hills and the anicut or dam, the river is 1300 yards wide, and has a depth in dry seasons of from 5 to 6 feet, in average freshes of 31, and in highest freshes of 38 feet. In the delta it runs on an elevated ridge, having an average fall to the sea of 1 foot per mile, varying from ·9 to 1·1 feet; the fall of the country on both sides towards the sea being 1·5 feet per mile. The irrigation of the delta, commenced by Captain Orr, provides for taking off 3500 cubic feet per second for each side of the river, but these works are still in an incomplete state; the irrigable area on each bank being capable of utilizing 32 000 cubic feet per second during the season of cultivation.

The tributaries of the Kistna.

The Tungabaddra, the most important tributary of the Kistna, has a length of about 213 miles from Gutal, where its upland tributaries, the Tunga, the Baddra, and the Choardi join the Warda, to its junction with the Kistna, at about 81 miles below Karnul. These four upland tributaries drain an area of 3754 square miles in the province of Maisur, a portion of which is hilly country, having a downpour of 135 inches, the remainder being plains with a downpour of only 24 inches.

Of these, the Warda, draining 610 square miles, has merely a few small anicuts on its feeders; its ordinary mansun discharge is roughly assumed to be 5000, and its maximum flood discharge 30 000 cubic feet per second. The Haggri—joined by its affluent, the Chinna Haggri, which falls into it near Mukalmuru—feeds the large Eyenkaira and Maddak tanks in a comparatively rainless district, and may eventually also supply an intended large reservoir at the Mauri Kunawai pass, where its discharge has been gauged for two years, giving as an ordinary mansun discharge 4500 and as a maximum flood discharge 50 000 cubic feet per second.

The Tunga, after being joined by the Baddra at Kudli, is joined by the Choardi at 10 miles above Harihar, and at Harihar itself by the Sulikerri; the maximum flood discharge of the combination of the three at the large bridge at Harihar has been determined to be 207 843 cubic feet per second, and the ordinary mansun discharge roughly calculated to be 30 000.

At Wallabapur, after a course of 55 miles, the Tungabaddra is joined by two tributaries, and at its 120th mile by the Haggri, after which it passes Sunkesala at its 175th mile, and Karnul before joining the Kistna. At Sunkesala are the headworks of a series of canals, flowing thence to Caddapa; and Wallavapur is the proposed site of headworks for a high-level canal, thence passing Ballari to Karnul. In order to afford further supply to these canals, it was proposed to enlarge existing reservoirs and make others on the upland tributaries of this river; and with this view some gaugings were made on them for six months, from June to November 1865, giving the following results:—

	Sq. miles.	Million cub. ft.	Inches run off.
The Tunga, at Shemuga ...	950	229 662	108
The Baddra, at Benkipur ...	884	125 928	63
The Choardi to Maddak tank	486	54 000	50 in floods.
The Haggri, at Heriur ...	1400	1 350	
The Tungabaddra at Wallabapur		356 940	
The Tungabaddra at Sunkesala		569 700	

The proposed reservoirs on the tributaries, intended to store the above supplies, and render the present Tungabaddra canals perennial, are the Mudaba on the Tunga, the Lakkawali on the Baddra, the Masur on the Choardi, and the Mauri Kunwai on the Haggri.

Further information about the upland tributaries is given among the tabular data of the rivers of Maisur.

The Penner rises in Maisur, about 150 miles above the Madras Railway-bridge, down to which point its catchment area is 4500 square miles. At Perur, where its upland tributaries have joined it, the channel is larger and becomes important; from this point its course is about 110 miles in length, without having any important tributary, to its junction with the Chittravatti above Jamalmagdu, where the catchment area of the latter stream is 3325 square miles: the maximum flood discharge of the Chittravatti is 23 100 cubic feet per second, and its ordinary mansun discharge is about one-tenth of that. About 40 miles below this its tributaries the Kunder and the Papagni rejoin it, the one having a catchment area of 3000, the other of 2460 square miles: the latter has a maximum flood discharge of 5244 cubic feet per second, and an ordinary mansun discharge of about one-tenth of that. At 32 miles below this the Sugaler and the Cheyer join it. At 18 miles below this, and at 70 miles from its debouchment into the sea, is Someshwaram, where the river leaves the Western Ghats, the site of the proposed headworks for a deltaic canal to irrigate the Nellor side of the delta. The total length of the river from Perur to the sea is about 270 miles. Its upland tributaries in Maisur are utilized (see tables of the rivers of Maisur), but for the rest of its course down to the head of the delta the river now flows on unimpeded. On the Kunder, at 25 miles above its junction with the Penner, is the Rajoli Dam and subsidiary headworks of the chain of canals from Sunkesala to Caddapa; the tributaries of the Kunder are also utilized in the same way, affording irrigation to the large valley of the Kunder.

For the greater part of the year the Penner, as low even as the Madras Railway bridge, is dry at the surface, though at from 1 to 4 feet in the bed plenty of water can always be found. The ordinary mansun floods are 6 to 8 feet deep; the extraordinary floods, 13 feet At the bridge-site the river is 1550 feet wide; the soil is clay for 5 feet, gravel mixed with clay and kunkur nodules for 4 feet more, resting on a layer of sand, superimposed on hard, dark green kunkur.

The Kaveri rises in the Western Ghats, and has a catchment area, together with its delta, of 32 000 square miles. It is fed by both mansuns, and its volume is abundant from the beginning of June to the end of December. The discharge on the 4th December, 1833, at the head of the delta, was 16 875 cubic feet per second, according to Col. Cotton: but in high flood the discharge is as much as 320 625 cubic feet per second. From January to May the discharge is small, much

less than 16 000 cubic feet per second; though there are freshets in March and April due to local storms. Above Seringham, in Tanjor, the Kaveri divides itself into the Kaveri and the Kalerun branches, which irrigate the delta, none of the water reaching the sea; this is due to the grand anicut of Seringham, constructed by the Telinghi rajas in remote antiquity, and restored and remodelled by Col. Cotton, between 1830 and 1836. The slope of the main stream above the bifucation is 3·5 feet per mile; from that to Seringham, that of the Kalerun is 2 feet per mile; from Seringham to the sea coast, its average slope is 1 foot per mile. The general fall of the main Kaveri branch is ·4 feet per mile less than that of the Kalarun. Before 1830, 12 622 cubic feet per second was utilized in irrigation from the Kaveri branch, and 4125 cubic feet per second from the Kalarun, or 16 474 cubic feet per second in all, out of 16 875. In 1833, the works constructed by Col. Cotton, utilized 9375 cubic feet per second from the Kaveri and 7500 from the Kalerun, the latter suffering as much from excess as the former from deficiency. In 1845 Col. Sim made a regulating dam across the head of the Kaveri, and lowered the Kalarun dam 2 feet, since when the regimen has been perfectly under control. The Kalerun is now not only a channel of irrigation, but is also the great drainage channel of the delta; the Kaveri is a channel of irrigation only, its entire volume being subdivided into small channels, and entirely utilized, although in its upper portion it is a mile in width. Information about these works is given under the head of the Kalerun deltaic canals.

The Tributaries of the Kaveri, consisting of the Upper Kaveri, the Somavatti, Hemavatti, Lachmantirth, and Lokani, join above Seringapatam. Their combined maximum flood discharge at Bannur, below that town, has been roughly determined to be 239 000 cubic feet per second; the ordinary mansun discharge, for a depth of 8 feet, is about 36 000 cubic feet per second. The other tributaries are the Kabbani, the Arkavatti, and the Shimsha; the maximum flood discharge of the Kabbani at Nanjengod is calculated to be 63 700 cubic feet per second, its ordinary mansun discharge about one-tenth of that; the maximum and ordinary mansun discharges of the Arkavatti at the Mangadi-road bridge are calculated to be 50 000 and 3500 cubic feet per second; the discharges of the Shimsha are assumed to be identical in quantity with the latter. Some further information about these tributaries is given in the data of the rivers of Maisur.

The Tambrapurni, rises in the Western Ghats, having its principal

source in the valley of Papanassan, drains a large tract of hilly and woodland country under the influence of both mansuns, and falls into the sea south of Tuticorin. Its catchment area is 200 square miles; its course for 20 miles is in forest covered mountains, where the rainfall is from 200 to 300 inches; and for 70 miles in plains at the foot of the hills, where the rainfall is from 20 to 30 inches; for the remainder of its course it receives a rainfall of only 18 inches. Its fall at Papanassan, and that of its tributary, the Chittar, at Kurtallam, are renowned for their beauty, and are considered sacred. There are seven native anicuts on the Tambrapurni, four on the Chittar, and two on the Mannemubuar: in addition to that now nearly constructed at Strivigantam by the English. Its floods commence in June, when they are sometimes 10 feet deep, and frequently recur during the next six months, or during the north-east mansun. The drainage from the hills keeps a hot weather stream, at Strivigantam, of about 314 cubic feet per second, and never less than 198 cubic feet per second in March; during the six months the discharge is not less than 600 cubic feet per second. The amount of its discharge utilized for irrigation is thus estimated in the Government records:—

	Cubic yards.
For 225 days of 1st crop at 32 cubic yds. per sec. =	237 600 000
For 45 days for 2nd crop at 15 cubic yds. per sec. =	58 320 000
For 45 days for 2nd crop at $7\frac{1}{8}$ cubic yds. per sec. =	28 382 400

Average depth at Strivigantam 7 feet, fall $2\frac{1}{2}$ to 3 feet per mile, velocity 5 to 5·6 feet per second.

The Upar.—The discharge of this stream has not been measured, nor are any observed velocities mentioned in the Madras government records, but its flood discharge has been thus approximated to by calculation. Its catchment area is 342 square miles, and it is supposed that there is a maximum rainfall in 24 hours of 8 inches over one-fourth of it, of 4 inches over another fourth, and of 2 inches over the remainder, and that the stream carries off one-fourth of this, three-fourths being lost by absorption and evaporation. This gives a flood discharge of 8850 cubic feet per second.

A LIST OF THE PRINCIPAL CANALS OF INDIA.

Perennial Canals in Northern India.

FULLY DEVELOPED.	Source.	Supply, actual or intended. C. ft. per sec.
The Western Jamna Canal	The Jamna ...	2372
The Eastern Jamna Canal	The Jamna ...	1068
HALF DEVELOPED.		
The Ganges and Lower Ganges Canals	The Ganges ...	5100
The Bari Doab Canal	The Ravi ...	2201

UNDERGOING REMODELLING.

Canals in the Dun and in Rohilkand.

UNDER CONSTRUCTION.

The Sarhind Canal	The Satlaj ...	3000
The Agra Canal	The Jamna ...	2000
The Orissa Canals	The Mahanaddi	various.
The Son Canal	The Son ...	5300
The Sakhar Canal	The Indus ...	unknown.

Inundation Canals in Northern India.

The Upper Satlaj Canals ...	aggregate length	224 miles.
The Lower Satlaj Canals ...	„	418 „
The Chenab Canals	„	222 „
The Jhelam Canals	„	unknown.
The Indus Canals in the Panjab	„	577 miles.
The Indus Canals in Sind ...	„	unknown.

Perennial Canals in Southern India.

	Supply.
The Tungabaddra Canals (not yet rendered perennial) ...	375

Inundation Canals in Southern India.

The *Deltaic* canals and anicuts of the Madras presidency.

Minor Canals in the Bombay presidency.

The anicuts and channels of Maisur.

Financial Statistics of Canals in operation in Northern India.

Abstract of General Results for 1872–73.

	Capital account to 1st April, 1873.	Gross income in 1872–73.		Working expenses in 1872–73.	Percentage of profit or loss.	1872–73.		No. of days the canal was open.
		Direct.	Total.			Acres irrigated.	Value of irrigated crops.	
THE PANJAB.								
Western Jamna Canal ...	£311 693	£95 362	£132 618	£40 118	31	351 820	£1 487 905	310
Bari Doab Canal	1 344 957	63 468	81 786	31 570	4	228 796	913 706	325
Upper Satlaj Canals	44 292	6 707	9 498	12 496	6	135 349	409 059	305
Lower Satlaj and Chenab ...	10 520	12 942	34 272	16 362	173	242 504	153 222	260
Indus Canals	43 736	7 866	15 960	18 046	—5	180 137	490 252	145
Shahpur Canals (Jhelam) ...	2 122	698	698	434	12	4 445	1 349	
Total	£1 757 320	£187 043	£274 832	£119 026	8·9	1 243 051	£3 455 493	
THE NORTH-WEST PROVINCES.							Charge for interest.	
Ganges Canal	£2 605 178	£158 992	£186 660	£98 871	3·4	685 170	£116 660	244
Eastern Jamna Canal	206 177	56 253	68 561	21 918	22·7	184 154	9 310	337
Dun Canals	57 253	4 791	5 265	2 504	4·9	14 002	2 544	...
Rohilkand Canals	103 601	2 438	5 699	5 132	·7	55 650	4 121	...
Total	£2 972 209	£222 474	£266 185	£128 426	4·7	938 976	£132 635	
BENGAL.		Navigation.						
The Orissa Series	£1 221 577	£1 004	£2 295	£23 822	—1·8	4 753	£80 213	...
Midnapur and Tidal Series ...	695 812	3 544	9 805	9 621	0·	13 406	41 202	...
Total	£1 917 389	£4 548	£12 100	£33 443	—1·8	18 159	£121 415	

Financial Statistics of Canals, Punjab.

Abstract of Results on the Western Jamna Canal.

Official year.	Capital Outlay.			Working Expenses.	Direct Revenue.	Indirect Revenue.	Total Yearly Return.	Percentage of Net Revenue on Capital.
	Original Works.	Establishment & other charges.	Total to end of year.					
	£	£	£	£	£	£	£	
to 1821	19 443	...	19 443	. 532	1 136	...	1 136	—
1821–22	...	...	...	2 568	111	...	111	—
1822–23	...	...	...	8 692	−1 065	...	−1065	−5
1823–24	...	...	...	5 056	− 514	...	− 514	−2
1824–25	...	...	...	5 756	−2 532	12 000	9 468	[illegible]
1825–26	414	41	21 894	6 861	− 945	12 000	11 055	57
1826–27	1 814	181	19 898	7 090	−2 875	12 000	9 125	47
1827–28	...	—	...	7 862	−2 583	12 000	9 417	47
1828–29	...	...	...	8 388	− 556	12 000	11 444	[illegible]
1829–30	494	49	22 498	8 479	− 219	12 000	11 781	54
1830–31	...	...	...	9 337	− 733	12 000	11 266	50
1831–32	...	...	...	10 206	−2 118	12 000	9 882	44
1832–33	...	...	...	10 797	−1 186	12 000	10 814	48
1833–34	7 201	720	80 359	9 982	7 701	12 000	19 701	88
1834–35	9 028	903	40 290	10 874	3 652	12 000	15 652	52
1835–36	62 222	6222	108 734	4 755	9 759	12 000	21 759	54
1836–37	338	34	109 106	9 439	9 642	12 000	21 642	20
1837–38	3 174	317	112 598	10 170	19 797	12 000	31 797	29
1838–39	4 604	460	117 663	8 227	13 913	12 000	25 913	23
1839–40	626	63	118 351	9 314	16 568	12 000	28 568	24
1840–41	3 118	312	121 781	9 224	19 634	12 000	31 634	27
1841–42	1 212	121	123 114	9 520	19 937	12 000	31 937	26
1842–43	2 512	251	125 877	10 847	20 279	37 256	57 535	47
1843–44	841	84	126 802	10 814	18 785	37 256	56 041	45
1844–45	264	26	127 092	16 927	9 104	37 256	46 361	37
1845–46	1 718	172	128 982	14 161	15 727	37 256	52 983	42
1846–47	5 677	568	135 227	13 196	17 092	37 256	54 348	42

Financial Statistics of Canals.—Panjab.

Abstract of Results on the Western Jamna Canal.

Official year.	Capital Outlay. Original Works.	Capital Outlay. Establishment & other charges.	Capital Outlay. Total to end of year.	Working Expenses.	Direct Revenue.	Total yearly Return.	Percentage of net Revenue on Capital.	Acreage irrigated.
	£	£	£	£	£	£		
1847–48	555	55	135 838	10 539	18 529	55 786	41	
1848–49	6 050	605	142 493	12 468	18 491	55 747	41	
1849–50	2 087	209	144 788	14 117	17 355	54 611	38	
1850–51	342	34	145 164	13 793	16 732	53 988	37	
1851–52	11 248	1 125	157 537	12 548	19 855	57 111	39	
1852–53	7 550	755	165 842	15 008	17 547	54 803	35	
1853–54	6 871	687	173 400	12 603	21 928	59 185	38	
1854–55	1 951	195	175 547	10 297	18 983	56 239	32	
1855–56	984	127	176 657	12 424	21 871	59 127	34	
1856–57	1 956	261	178 874	16 938	9 386	46 642	26	
1857–58	491	81	179 446	10 664	12 754	50 011	28	
1858–59	1 838	261	181 545	16 313	16 632	53 888	30	
1859–60	2 222	330	184 096	20 317	16 316	53 573	30	
1860–61	3 721	493	188 310	21 865	24 470	61 726	33	454 292
1861–62	8 906	1 185	198 401	22 250	18 147	55 404	29	372 680
1862–63	4 096	1 449	203 945	17 426	17 586	54 843	28	303 361
1863–64	6 845	4 618	215 408	16 408	23 297	60 553	30	351 537
1864–65	10 019	476	225 904	21 179	—5 710	31 547	15	434 964
1865–66	903	859	227 666	20 285	28 477	65 733	20	397 963
1866–67	446	304	228 417	23 150	34 229	71 485	31	447 171
1867–68	1 795	364	230 577	28 711	66 313	103 569	45	331 037
1868–69	10 716	5 696	246 989	24 102	39 574	76 830	33	486 878
1869–70	7 939	7 955	262 884	38 979	74 405	111 611	45	496 542
1870–71	4 816	11 474	279 173	33 873	116 884	154 140	59	462 707
1871–72	5 780	13 084	298 036	37 645	71 651	108 907	39	444 385
1872–73	3 454	9 895	311 693	40 118	62 182	99 438	33	351 821

Financial Statistics of Canals.—North-West Provinces.

Abstract of Results on the Eastern Jamna Canal.

Official year.	Capital Outlay. Original Works.	Capital Outlay. Establishment & other charges.	Capital Outlay. Total to end of year.	Working Expenses.	Direct Revenue.	Indirect Revenue.	Total Yearly Return.	Percentage of net Revenue of Capital.	Acreage irrigated.
	£	£	£	£	£	£	£		
1823 to 1830–31	31 124	12 676	43 800						
1830–31 to 1846–47	49 074	4 907	97 781	97 522	21 454				
1847–48	1 435	143	99 360	6 904	12 503	14 965	...	...	106 705
1848–49	3 254	325	102 939	7 042	15 055				
1849–50	3 460	346	106 745	8 016	16 183				
1850–51	304	30	107 079	7 392	15 914				
1851–52	2 558	256	109 893	7 726	13 079				
1852–53	3 057	306	113 256	8 279	17 325				
1853–54	5 315	531	119 102	7 872	14 993				
1854–55	16 376	1 688	137 665	9 565	14 479				
1855–56	12 691	1 637	151 994	8 188	9 688				
1856–57	5 180	691	157 865	13 540	12 997				
1857–58	1 351	223	159 440	7 691	6 645				
1858–59	2 260	337	162 036	9 255	12 483	...	...	...	154 006
1859–60	393	81	162 510	10 575	20 924	...	...	...	227 489
1860–61	973	141	163 624	11 376	28 941	...	...	...	261 327
1861–62	603	3 071	167 298	11 305	22 873	...	...	...	231 310
1862–63	1 346	—300	168 343	8 518	25 696	3 800	29 496	13	184 232
1863–64	1 218	1 732	171 283	10 799	23 217	6 000	29 217	11	181 331
1864–65	3 366	432	174 981	12 518	36 539	6 000	42 539	18	225 266
1865–66	2 876	1 612	179 469	13 061	41 463	6 000	47 463	20	160 355
1866–67	2 844	2 269	184 582	12 247	43 131	17 769	60 900	27	239 555
1867–68	4 930	1 816	191 328	14 208	56 560	17 769	74 329	33	182 544
1868–69	4 904	1 246	197 479	15 488	50 624	17 760	68 393	26	274 101
1869–70	2 779	282	200 539	16 508	65 728	17 769	83 497	34	251 067
1870–71	2 324	303	203 166	18 096	60 104	17 769	77 873	30	212 715
1871–72	1 973	—204	204 935	19 880	51 026	17 769	68 795	24	192 749
1872–73	1 895	—654	206 177	21 918	56 254	12 307	68 561	23	184 154

Financial Statistics of Canals in the Panjab.

Capital account of the Western Jamna Canal, to the end of 1872-73

Detail.	Previous.	In 1872-73	Total.
WORKS.	£	£	£
A. *Temporary Works* (to maintain supply)	...	78	78
B. *Cost of Land*	3 316	29	3 345
C. *Masonry Works.*—1. Main Canal and branches	...	...	...
a Dams, and regulating works ...	2 487	1 017	3 504
b. Falls and weirs...	9 050	336	9 387
c. Aqueducts	248	...	248
d. Escapes	563	...	563
4. Supply of tanks...	1 555	...	1 555
5. Road bridges	1 679	...	1 679
8. Buildings	201	330	350
D. *Earthwork.*—1. Main Canal and branches....	18 542	948	19 490
3. Drainage works	1 714	...	1 714
E. *Miscellaneous.*	1 312	138	1 450
Total Main Canal, and branches ...	40 486	2 877	43 364
Distributing Channels.			
C. *Masonry works.*—*d.* Irrigation Outlets	...	576	576
Expenditure on general works up to 1863-64	194 341	...	194 341
Total on WORKS ...	234 827	3 453	238 281
ESTABLISHMENT, GENERAL.			
Direction	...	908	...
Executive	...	4 430	...
Survey	...	5 417	...
Total on ESTABLISHMENT ...	56 645	10 755	67 400
TOOLS AND PLANT.			
Total on Tools and Plant ...	1 407	19	1 426
	292 879	14 228	307 107
Add or deduct fluctuations of suspense balance: for stock, sales, and advances ... ... Total	5 158	−572	4 586
Total CAPITAL OUTLAY. £	298 037	13 656	311 693

Financial Statistics of Canals in the Punjab.

Capital account of the Bari Doab Canal to the end of 1872-73.

Items.	Previous.	In 1872-73.	Total.
Works.	£	£	£
B. *Cost of Land*	7 333	...	7 333
C. *Masonry works*.—1. Main Canal and branches			
a. Dams and regulating works ...	75 794	155	75 949
b. Falls and weirs	138 242	5 675	143 917
c. Aqueducts	17 363	...	17 363
d. Escapes	15 474	...	15 474
2. Drainage works	2 473	...	2 473
3. Road bridges	103 601	94	103 694
6. Navigation works	18 949	...	18 949
7. Mills	1 267	...	1 267
8. Buildings	22 014	536	22 550
D. *Earthwork*			
1. Main Canal and branches ...	432 778	3 823	436 601
2. Drainage works	7 101	...	7 101
3. Navigation Channels	8 193	...	8 193
E. *Miscellaneous*	65 736	46	65 782
F. *Plantations*	5 507	...	5 507
Total Main Canal and branches	921 276	11 398	932 674
Distributing Channels			
B. *Cost of Land*	3 567	...	3 567
C. *Masonry works*, a, Head sluices and regulating works	5 343	113	5 456
b. Falls and weirs	11 194	...	11 194
c. Aqueducts	14 032	...	14 032
d. Irrigation outlets	6 113	815	6 928
D. Earthwork	73 967	243	74 210
Total on Works ...	1 035 492	12 569	1 048 061
Establishment, General.			
Direction	...	1 761	...
Executive	...	11 354	...
Medical	...	51	...
Total Establishment ...	202 715	13 166	215 881
Tools and Plant	46 853	70	46 923
Profit and loss	4 477	...	4 477
Fluctuations of suspense-balance...	29 592	23	29 615
Total Capital Outlay ...	1 319 129	25 828	1 344 957

Financial Statistics of Canals in the North-West Provinces.

Capital account of the Eastern Jamna Canal, to the end of 1872–73.

Detail.	Previous.	In 1872-73.	Total.
	£	£	£
WORKS.			
Main Canal.			
C. *Masonry works.* Syphon ...	...	14	14
Bridge	103	583	686
Buildings		182	182
D. *Earthworks.* Canal banks ...		49	49
Drainage works—sheds	545	44	590
Other works	158 737	...	158 737
Total Main Canal ...	159 385	872	160 257
Distributing Channels.			
The cost of these is not shown, they were made by the cultivators ...			
C. *Masonry works*		683	683
D. *Earthwork*		120	120
Escape	45	220	265
Other works	8 936	...	8 936
Total on WORKS ...	168 366	1 895	170 261
ESTABLISHMENT.			
Direction	2 328	180	2 508
Executive	26 600	250	26 850
Total on ESTABLISHMENT ...	28 928	430	29 358
Tools and Plant	621	42	663
Profit and loss	20	...	20
Fluctuations of suspense balance	7 000	—1 119	5 881
Less Receipts		—6	—6
NET OUTLAY ...	204 935	1 242	206 177
Add Simple Interest	243 272	9 310	252 582
Total CAPITAL OUTLAY ...	448 207	10 552	458 759

12

Financial Statistics of Canals in the North-West Provinces.

Capital account of the Ganges Canal, to the end of 1872–73.

Detail.	Previous.	In 1872–73.	Total.
WORKS.	£	£	£
Head works			
C. *Masonry works.* Weirs		2 457	2 457
Main Canals and branches ...			
B. *Cost of Land*		8	8
C. *Masonry works.* Falls and Weirs		8 559	8 559
Bridges		11 894	11 894
Navigation works		534	534
Buildings		289	289
D. *Earthworks.* Canal embankments, &c.		620	620
F. *Miscellaneous.* Loss on bricks ...		1 557	1 557
Escapes		1 077	1 077
Drainage works		1 856	1 856
Other works (?)	1 698 817	...	1 698 817
Total Main Canal and branches ...	1 698 817	28 851	1 727 668
Distributing Channels.			
Preliminary operations		234	234
B. Cost of Land		944	944
C. Masonry works		4 570	4 570
D. Earthworks		4 155	4 155
Other works (?)	450 169	...	450 168
Total on WORKS ...	2 148 986	38 754	2 187 740
ESTABLISHMENT.			
Direction	55 081	1 615	56 696
Executive	232 302	3 866	236 168
Remodelling	16 671	...	16 671
Total on ESTABLISHMENT ...	304 054	5 481	309 535
Tools and Plant	16 725	1 473	18 199
Profit and Loss	7 101	...	7 101
Fluctuations of suspense-balance...	109 146	−17 153	91 993
Less Receipts	−9 282	− 107	−9 389
NET OUTLAY ...	2 576 730	28 448	2 605 178
Add Simple Interest	1 941 670	116 660	2 058 330
Total CAPITAL OUTLAY ...	4 518 400	145 108	4 663 508

Financial Statistics of the Deltaic Canals of Southern India.

Abstract of approximate results from remunerative works of irrigation, anicuts and channels, exclusive of tanks, in the Madras presidency.

Name of District.	Name of Anicut.	Up to end of 1872–73		For year 1872–73.		Percentage of net profit.
		Total Capital Outlay.	Total Gross Income.	Interest & Maintenance.	Gross Proceeds.	
		£	£	£	£	
Godavari ...	Godavari ...	544788	3 427 377	36 023	214304	32·7
Kistna ...	Kistna	358254	782 199	24 669	69303	12·5
Nellor ...	Pennar ...	93395	89 142	6 200	8954	2·9
Chinglipat...	Four anicuts ...	12411	32 133	743	8846	63·2
Chinglipat...	Palar	21493	23 233	955	5723	
North Arcot	Palar	75086	34 139	3 718	2648	
Total	*Palar*	96579	57 372	4 673	8371	3·8
North Arcot	Poini	15420	34 987	702	641	loss
North Arcot	Alliabad and Cheyar	20207	24 450	1 407	2542	5·5
South Arcot	Vellar and nine others	52055	395 809	4 961	33321	53·8
South Arcot	Lower Kalerun	12974	1 106 873	2 399	41193	
Tanjor	Lower Kalerun	43974	66 118	1892	1967	
Tanjor	Upper Kalerun	24066	1 757 088	1 165	67083	
Total	*Kalerun* ...	81014	2 930 079	5 456	110243	128·3
Trichinopoly	Nandiar ...	7855	9 640	406	944	6·8
Coimbator...	Four channels...	22961	24 288	3 216	2844	loss
Malabar ...	Yenamakal ...	4250	5 408	296	141	loss

N.B.—The capital outlay does not include deduction for wear and tear, nor, in some instances, the cost of the distributaries. The interest is 4 per cent. on the outlay up to the beginning of 1872–73.

Financial Statistics for 1864–65 *of the Anicuts and Channels of Maisur.*

Division.	Rivers utilized.	Aggregate length of Channels.	Revenue realized in 1864–65.
		Miles.	£
I.—Maisur	Kaveri, Lachmantirth, Shimsha, Nugu	461	24 025
II.—Hassan	Kaveri, Himavatti, Yegachi, its branches, Shimsha	232	5 910
III.—Kaddur	Vadvutti, Biranji, Kirisandisamudram	148	3 456
IV.—Naggar	The tributaries of the Tungabaddra	362	3 791
	Total ...	1203	37 182

I.—Abstract for the Maisur Division.

Name of Anicut.	Length of Channel.	Measured Discharge.	Irrigable area at a duty of 40 acres.	Assessment due at the rate 15s. per acre.	Revenue realized in 1864–65.
From the Kavery.	Miles.	C. ft. p. sec.	Acres.	£.	£.
Saligram	13	40	1 600	1 200	717
Mirlao	40	151	6 060	4 545	1 924
Chanchamcattai ...	24	123	4 920	3 690	1 212
Tippur	22	83	3 320	2 490	616
Chikdeoraj	75	448	17 920	13 440	6 070
Davroi	8	73	2 920	2 190	468
Vijjianaddi	35	240	9 600	7 200	3 262
Bangardodi	9	90	3 600	2 700	758
Ramasami	31	118	4 720	3 540	2 369
Do.	30	118	4 720	3 540	1 287
Talkad	18	153	6 120	4 590	1 288
From the Lachmantirth.					
Hanagod	17	335	13 400	10 050	1 211
Kattai Malwadi ...	14	140	5 600	4 200	239
Harganhalli	12	150	6 000	4 500	237
Do.	17	224	8 960	6 720	289
Sagar	20	...	...	...	498
Cholenhalli	6	...	...	...	148
From the Shimsha.					
Maddur	12	56	2 240	1 680	728
From the Nugu.					
Lachmanpura	4	135	5 400	4 050	704
Total	461	2677	107 100	80 325	24 025
Averages per cubic ft. per second of discharge ...		1	40	£ 30	£ 9

Financial Statistics for* 1864–65 *of the Anicuts and Channels of Maisur—continued.

II.—Abstract for the Hassan Division.

Name of Rivers.	Number of Anicuts.	Number of Channels.	Length of Channels.	Revenue realized in 1864–65.
			Miles.	£.
Yegachi	...	4	15¼	472
Kaveri	...	2	53	2010
Himavatti	...	8	112¾	2821
Branch of Yegachi ...	...	4	46	588
Shimsha	...	1	5	19
Total ...	...	19	232	5910

III.—Abstract for the Kaddur Division, including Chikmaglur.

Names of Rivers.	Number of Anicuts.	Number of Channels.	Length of Channels.	Revenue realized in 1864-65.
			Miles.	£.
Vedavatti	56	75	120½	3086
Billah	1	1	1½	23
Biranji	6	6	13½	340
Kirisandisamudram ...	1	...	2	7
Total ..	64	82	138	3456

IV.—Abstract for the Naggar Division, Shemogah and Kaddur.

District.	River System.	Number of Anicuts.	Length of Channels.	Revenue realized in 1864-65.
			Miles.	£.
Sagar ...	Sheravatti ...	46	8½	878
	Warda	22	14	
Naggar	Sheravatti ...	19	6½	75
Kaulidrug ...	Tunga	7	6	69
Lakawali	Baddra	15	107¼	518
	Tunga	2	7	
Surab	Warda	22	17	406
Shikarpur	Choardi	8	25½	183
	Warda	3	4	
Shemogah ...	Tunga	22	63	900
Honnahalli ...	Tungabaddra ...	3	...	22
Terrikerrai ...	Baddra	4	2¼	5
Anantapur	Warda	4	5¼	135
	Choardi	4	8	
	Sheravatti ...	5	11	
Wastara ...	Biranji	64	77½	600
	Total ...	250	362	3791

Statistics of Irrigation from the Western Jamna Canal.

Year.	Supply admitted.	Supply utilized.	Acreage Irrigated.			Length of Distributaries.	Rainfall.
			Kharif.	Rabbi.	Total.		
	C. ft. p. sec.	C. ft. p. sec.				Miles.	Inches.
1872–73	2125	1802	202 370	149 450	351 820	The distributaries chiefly belong to the landholders.	46 to 20
1871–72	2147	1928	187 647	256 738	444 385		70 to 11
1870–71	2067	1797	218 535	244 172	462 707		48 to 15
1869–70	2372	...	234 465	262 078	496 542		31 to 6
1868–69	2277	...	198 670	288 208	486 878		31 to 9
1867–68	1499	...	186 887	144 150	331 037		31 to 9
1866–67	1833	...	211 103	236 068	447 171		68 to 23
1865–66	1615	...	196 271	201 692	397 963		37 to 7
1864–65	1800	...	197 673	237 291	434 964		
1863–64	1254	...	...	...	351 537		

The area of double cropped land is about 13 per cent. of the total acreage.

Irrigating capacity varied from 430 500 acres in 1864 to 536 580 in 1871.

Mileage of canal open from 1860 to 1873—Main 102 ; branches, 313.

Statistics of Irrigation from the Eastern Jamna Canal.

Year.	Supply admitted.	Supply utilized.	Acreage Irrigated.			Length of Distributaries.	Rainfall.
			Kharif.	Rabbi.	Total.		
	C. ft. p. sec.	C. ft. p. sec.				Miles	Inches.
1872–73	1050	998	79 699	104 445	184 154	625	74 to 25
1871–72	981	95 p.c.	72 404	120 345	192 749	610	114 to 27
1870–71	956	98 p.c.	98 112	114 603	212 715	608	
1869–70	...	100 p.c.	119 163	131 904	251 067	606	
1868–69	...	98 p.c.	102 141	171 960	274 101	603	
1867–68	...	94 p.c.	78 606	103 938	182 544	596	
1866–67	1068	100 p.c.	82 138	157 417	239 555	596	
1865–66	...	...	80 225	80 130	160 355	596	
1864–65	1025	...	117 770	117 770	225 266	602	
1863–64	932	...	71 129	110.202	181 331	602	
1862–63	1043	...	...	...	184 232	602	

Irrigating capacity, 1858 to 1873—250 000 acres.

Mileage of main canal, 1862 to 1873—130 miles.

Statistics of Irrigation from the Bari Doab Canal.

Year.	Supply admitted.	Supply utilized.	Acreage Irrigated. Kharif.	Acreage Irrigated. Rabbi.	Acreage Irrigated. Total.	Length of Distributaries.	Rainfall.
	C. ft. p. sec.	C. ft. p. sec.				Miles.	
1872–73	1838	1208	96 718	132 078	228 796	716	
1871–72	2073	1950	76 412	210 658	287 079	712	
1870–71	2201	2069	88 643	190 567	279 210	710	
1869–70	1948	1578	115 524	118 403	233 927	710	
1868–69	1899	1649	85 519	214 315	299 834	706	
1867–68	1532	...	106 043	156 085	262 128	696	
1866–67	1688	...	92 699	135 753	228 452	671	
1865–66	1431	...	91 378	84 602	175 980	623	
1864–65	1228	...	66 370	126 313	192 683	581	
1863–64	1340	1193	64 195	70 167	134 362	554	
1862–63	1450	...	59 476	66 540	126 016	409	
1861–62	1387	...	...	...	134 362		

The area of double cropped land from 1870 to 1873 was 8 per cent. of the whole acreage.

Mileage of canal, from 1860 to 1873. Main, 140 miles; branches, 59 miles.

Statistics of Irrigation from the Ganges Canal.

Year.	Supply admitted.	Supply utilized.	Acreage Irrigated. Kharif.	Acreage Irrigated. Rabbi.	Acreage Irrigated. Total.	Length of Distributaries.	Rainfall.
	C. ft. p. sec.	C. ft. p. sec.				Miles.	Inches.
1872–73	4787	4221	247 191	437 979	685 170	3228	33
1871–72	4191	76 p.c.	232 688	373 867	606 555	3078	36
1870–71	4300	89 p.c.	266 683	499 931	766 614	3071	38
1869–70	5100	90 p.c.	341 846	438 560	780 406	3069	28
1868–69	4946	94 p.c.	344 267	734 132	1 078 399	3112	16
1867–68	3952	86 p.c.	185 137	348 319	533 456	3040	46
1866–67	3940	89 p.c.	181 658	453 076	634 734	3039	26
1865–66	4314	...	176 544	396 585	573 129	2777	
1864–65	4026	...	161 835	404 682	566 517	2440	
1863–64	4028	...	97 538	352 250	449 788	2337	
1862–63	4850	...	90 693	114 912	205 605	2266	

Mileage of canal, 1862 to 1873. Main, 519 miles; branches, from 1862 to 1866, 127 miles; 1867 to 1873, 135 miles.

Irrigating capacity, 1 205 000 acres, during the above period.

Statistics of Irrigation of the Western Jamna and Bari Doab Canals for 1872-73.

Statement of Water utilized.

Western Jamna Canal.				Bari Doab Canal.			
	Supply at head.	Discharged from escapes.	Utilized.		Supply at head.	Discharged from escapes.	Utilized.
	Cub. ft. per sec.	Cub. ft. per sec.	Cub. ft. per sec.		Cub. ft. per sec.	Cub. ft. per sec.	Cub. ft. per sec.
Kharif. 1872.				*Kharif.* 1872.			
April ...	2359	234	2125	April ...	2198	1060	1138
May ...	2523	555	1968	May ...	2208	1046	1162
June ...	2446	288	2158	June ...	2146	504	1642
July ...	2319	229	2090	July ...	1776	850	926
August ...	2142	562	1580	August ...	1796	768	1028
September	1620	143	1477	September	1986	561	1425
Average	2234	335	1899	Average	2018	798	1220
Rabbi. 1872.				*Rabbi.* 1872.			
October ...	2413	353	2060	October ...	2202	989	1213
November	2540	374	2166	November	2095	915	1180
December	1941	396	1545	December	1640	471	1169
1873.				1873.			
January ...	1242	341	901	January ...	782	217	565
February ..	1872	249	1623	February ..	880	49	831
March ...	2084	152	1932	March ...	2342	125	2217
Average	2015	311	1704	Average	1657	461	1196
Average of year	2125	343	1802	Average of year	1838	629	1208

Approximate Acreage of the Irrigated Crops of the Western Jamna Canal in 1872–73.

Crops.	Kharif. Flow.	Lift.	Total.
Class 1.			
Sugar-cane...	...	unknown	unknown
Total ...	42 034	1 260	43 294
Class 2.			
Gardens ...	...	unknown	
Rice	...	...	43 143
Total ...	44 281	391	44 672
Class 3.			
Cotton ...	90 210	5 919	96 129
Hemp... ...	...	...	305
Tobacco ...	...	...	170
Waternuts...	...	...	unknown
Kachra ...	...	...	15
Sesamum ...	...	...	25
Toria	...	...	114
Indigo ...	6 317	172	6 489
Miscellaneous vegetables	...	unknown	unknown
Total ...	96 646	6 261	102 907
Class 4.			
Jowar	4 021	182	4 203
Kangni ...	41	2	43
Bajra	312	35	347
Maize	698	195	893
Samvak ...	[illegible]	...	2
Mundwa ...	237	68	305
Urad	404	6	410
Mung	4	...	4
Moth	74	3	77
Lucerne ...	56	85	141
Grass	28	2	30
Miscellaneous	unknown	unknown	unknown
Total ...	6 880	4 617	11 497
Class 5.			
Fallow ...	...	4 069	4 069
Total Kharif	189 840	12 530	202 370

Crops.	Rabbi. Flow.	Lift.	Total.
Class 1.			
...	...	unknown	unknown
Total ...	1 001	236	1 237
Class 2.			
...	...	unknown	unknown
Total ...	3 700	1 786	5 486
Class 3.			
Wheat ...	84 691	8 908	93 599
Barley ...	3 282	320	3 602
Oats	19	...	19
Toria	467	61	528
Tobacco ...	346	755	1 101
Poppy	5	1	6
Coriander ...	479	384	863
Methi	23	...	23
Other grains	16 848	338	17 186
Miscellaneous	...	unknown	
Total ...	115 098	10 105	125 202
Class 4.			
Masur	1 520	159	1 679
Chena	45	51	96
Charri	183	10	193
Javi	210	20	230
Lucerne ...	111	61	172
Grass	16	...	16
Miscellaneous	...	unknown	unknown
Total ...	244	17 280	17 524
Class 5.			
Gram	7 704	92	7 796
Fallow ...	...	2 231	2 231
Floodings ...	...	10 485	10 485
Total Rabbi	120 044	29 406	149 450
Kharif ...	189 840	12 530	202 370
Grand total .	309 884	41 936	351 820

N.B.—The totals of classes are correct; the detailed acreages are evidently incorrectly classified in several instances, the crops under Classes 2 and 5 being disseminated.

Acreage of the Irrigated Crops of the Eastern Jamna Canal in 1872–73.

	Crops.		Kharif.			Rabbi.		
		Class.	Flow.	Lift.	Total.	Flow.	Lift.	Total
	Gardens	2	1 006	242	1 248			
		2	...	...	...	1 279	175	1 454
	Sugar-cane ...	1	22 711	4 715	27 426			
Cereals.	Rice ...	4	37 616	135	37 751			
	Maize ...	4	2 077	499	2 576			
	Mundwa ...	4	28	2	30			
	Sanwak ...	2	9	...	9			
	Jowar ...	2	160	19	179			
	Chena ...	4	28	7	35			
	Wheat ...	3	...	...	...	70 814		
	Oats ...	3	...	...	...	883	18 580	89 394
	Gram ...	3	...	...	...	2 379	26	909
	Barley ...	3	...	...	...	2 645	661	3 040
Pulses.	Urd ...	4	28	7	35		551	3 196
	Moth ...	4	3	2	5			
	Masur ...	3	...	...	...	1 274		
	Peas ...	3	...	...	...	2 146	149	1 423
	Arhar ...	3	...	...	...	2	560	2 706
Fodder.	Lucerne	2	159	24	183		...	2
		2	...	...	...	117		
	Dumera	2	11	...	11		27	144
		2	...	...	...	16		
	Charri ...	2	215	24	239		...	16
Fibres.	Cotton ...	3	5 936	896	6 832			
	San ...	4	3	...	3			
Drugs & Dyes.	Indigo	3	1 963	239	2 202			
		4	...	...	...	2		
	Tobacco	2	92	40	132		...	2
		2	...	...	...	20		
	Opium ...	2	...	...	...	2	1	21
	Linseed ...	3	...	...	...	51	...	2
	Mustard ...	3	...	...	...		1	52
	Waternuts ...	2	54	...	54	1 456	161	1 616
	Waste	4	659	90	749			
	Irrigation	4	...	...	...	434	43	477
	Totals ...		72 758	6 941	79 699	83 520	20 935	104 455
			83 520	20 935	104 455			
	Grand Total ...		156 278	27 876	184 154			

Acreage of the Irrigated Crops of the Bari Doab Canal in 1872–73.

Crops.	Kharif.		
	Flow.	Lift.	Total.
Class 1.			
Sugar-cane...	8 688	470	9 158
Class 2.			
Gardens ...	225	6	231
Rice	28 829	264	29 093
Total ...	29 054	270	29 324
Class 3.			
Orchards ...	780	76	856
Cotton ...	24 879	2 658	27 537
Hemp	169	9	178
Sesamum ...	461	25	486
Miscellaneous	1 314	457	1 771
Total ...	27 604	3 225	30 829
Class 4.			
Jowar	9 158	1 341	10 499
Kangni ...	1 434	424	1 858
Maize	7 762	791	8 553
Miscellaneous	5 300	132	5 432
Fallow... ...	1 049	14	1 063
Total ...	24 705	2 702	27 407
Total Kharif.	90 051	6 667	96 718
Total Rabbi .	115 683	16 394	132 077
Total ...	205 735	23 061	228 796

Crops.	Rabbi.		
	Flow.	Lift.	Total.
Class 1.			
Sugar-cane...	11	1	12
Class 2.			
Gardens ...	117	7	124
Rice	7	...	7
Total ...	124	7	131
Class 3.			
Orchards ...	820	53	873
Wheat ...	84 866	14 751	99 417
Barley... ...	4 291	164	4 455
Linseed ...	22	3	25
Saru	731	226	957
Safflower ...	...	...	...
Poppies ...	475	58	533
Tobacco ...	56	13	69
Tukhmbalanga	...	...	...
Miscellaneous	415	262	677
Total ...	91 476	15 530	107 006
Class 4.			
Gram	7 370	78	7 448
Masur... ...	174	1	175
Sinji	14 182	686	14 868
Fallow ...	768	17	785
Miscellaneous	1 578	72	1 651
Total ...	24 072	855	24 927
Total Rabbi .	115 683	16 394	132 077

Acreage of the Irrigated Crops of the Ganges Canal in 1872–73.

	Crops.	Class.	Kharif. Flow.	Kharif. Lift.	Kharif. Total.	Rabbi. Flow.	Rabbi. Lift.	Rabbi. Total.
	Garden produce ...	2	3 198	1 752	4 950	2 974	1 790	4 764
	Sugar-cane ...	1	51 526	16 871	68 397	19	5	24
Cereals.	Wheat ...	3	13	16	29	168 592	73 897	242 409
	Barley ...	3	9	26	35	77 071	75 590	152 661
	Oats ...	3	...	...	...	80	8	88
	Rice ...	2	26 178	584	26 762	124	157	281
	Maize ...	4	354	224	578	...		
	Jowar ...	3, 4	835	180	1 015	...		
	Chena ...	3, 4	212	222	434	40	28	68
	Marna ...	4	921	906	1 827	...		
	Miscellaneous	3, 4	2 153	997	3 150	1 701	589	2 290
Pulses.	Gram ...	3	15	3	18	13 326	3 872	17 198
	Peas ...	3	...	...	...	4 938	1 767	6 705
	Arhar ...	4	...	...	...	37	...	37
	Masur ...	3	...	...	...	38	15	51
	Miscellaneous	3, 4	11	1	18	1 728	67	1 795
Fodder.	Charri ...	4	124	2	126			
	Lucerne ...	3, 4	87	25	112	75	15	90
	Miscellaneous	3, 4	17	1	18	1 477	87	1 564
Fibres.	Cotton ...	3	6 722	1 239	7 961			
	San	4	199	60	259			
	Flax	3	...	...	...	114	18	132
	Miscellaneous	3, 4	778	11	789	146	10	156
Drugs and Dyes.	Indigo ...	3	97 267	31 513	128 780	1	...	1
	Miscellaneous	3, 4	586	16	602	18	3	21
	Opium ...	2	10	21	31	2 094	4 636	6 730
	Tobacco ...	2	76	78	154	41	77	118
	Miscellaneous	3, 4	...	...	...	134	162	296
	Oilseeds ...	3, 4	...	...	...	141	2	143
	Waternuts ...	2	3	...	3	1	...	1
	Waste irrigation ...	4	648	495	1143	144	132	276
	Totals ...		191 948	55 243	247 191	275 054	162 925	437 979
			275 054	162 925	437 979			
	Grand Total ...		467 002	218 168	685 170			

Brief Accounts of Indian Canals.

The Western Jamna Canal is the oldest of the perennial canals of Northern India, the most fully developed as regards its powers of irrigation, and the most remunerative. It has, however, been carried on in a most desultory manner, and even now is not complete. In 1821, the capital expended on it was £14 216, and from that time to 1833 the progress was next to nothing; in 1835, the capital account was £33 168; but in 1836, £62 225 were spent, raising it to £100 000; from that time to 1846 next to nothing was spent, the account at that date being only £119 405, according to the returns formerly given. The present capital account, given in the accompanying statistics, gives different figures, owing to an entirely new arrangement; but the same rate of carrying on the works is clearly illustrated by them. In 1853–54, this canal had arrived at a very good stage of development, after more than thirty years had been passed in spending £175 000 on works. Up to 1872–73, the capital account was £311 693, but even yet the canal has no permanent headworks, and the drainage works necessary for the healthy control of the irrigation can only be said to be commenced; and half a century has elapsed since the British first took the matter in hand.

The canal is of Musalman origin, having been projected and carried out on a small scale under the Mughal emperors. Its head is at Tajawalla, on the west bank of the Jamna, 13 miles above Dadupur; the supply being conducted from the head along an old branch of the Jamna to Bhilpur, thence by an artificial cut into the Pattrala hill torrent, and then along the latter, down to a junction with the Sombe torrent near Dadupur, where a dam and regulating head for the supply of the actual main canal are situated. After 102 miles of main canal, it divides itself at Rer, into two main branches, the Delhi branch, 75 miles long, tailing into the Jamna near Delhi, and having distributaries aggregating 100 miles in length, and the Hansi branch, which is 108 miles long to Mingnikhera, and has 141 miles of distributaries, in addition to its sub-branches. At the Joshi regulator, in the 11th mile of the Hansi branch, is the head of a sub-branch, which loses itself in the sandy desert near Rohtak after a course of 43 miles. At the 13th mile of the Hansi branch, is the head of the Butana

sub-branch, 18 miles long, down to its bifurcation into two channels, one 11 the other 6 miles long.

At Minginkhera, the 108th mile of the main canal, is the head of the Bahadura sub-branch, 32 miles long, and of the Darba sub-branch, which is 18 miles long down to its bifurcation at Ramsira, whence it becomes two channels, each 10 miles long.

In addition to the various branches and distributaries, there are escape cuts from the main canal amounting to 55 miles in length, and 62 miles of escapes, cuts, and drainage lines from the Delhi branch. It is also proposed to make a new branch from the 59th mile of the main canal to Bhowani.

As regards the width of the canal, the main line varies from 360 to 120 feet, and the branches from 100 to 10; the depth is variable, the full supply depth at Dadupur being 4·3 feet, and the lowest supply about half of that,—the velocity at Tajawalla is about 17, and at Dadupur with full supply 4·14 feet per second.

The tract irrigated is 120 miles by 10.

In 1837-38, a year of famine, the acreage irrigated was 306 000, the produce saved being valued at £1 462 800; and the estimated value of the irrigated crops on 351 820 acres in 1872-73, being £2 021 811. In 1846-47, 351 501, or (360 902 ?) acres were actually watered, and the following works were completed; main canal 445 miles, excluding distributaries; bridges of various sorts, 240; main headworks, 1; stop dams, 12; aqueducts, 2; weirs and falls, 9; escapes, 4; locks, 2; irrigation outlets, 672; inlets, 36; station houses, 88; besides depôts, mills, and workshops. The gross returns in 1846 amounted to 55 per cent. on the capital. The irrigating power of water on this canal is higher than that of any canal in India, having sometimes reached nearly 300 acres per cubic foot per second of supply utilized.

While the Western Jamna canal yields the most favourable results as regards its powers of irrigation, this appears rather to be due to natural conditions than to skilful management. In 1819-20, before British reconstruction, the tract irrigated, 992 square miles, yielded £200 655 in water rate, while in 1850-51, the tract irrigated was 1015 square miles, yielding £242 177 in water rate; the increase of land revenue in each case amounting to £41 521, and the advantages due to British military management over a quarter of a century appearing very small in this particular.

The capital account of this canal was altered in the year 1863-64, by debiting it from 1820 with a share of expenses for establishment and contingencies, thus changing the sum from £190 404 to £212 899

on 1st May, 1864:—there is also some doubt about the establishment charges, whether they should be 10 or 13 per cent. on the cost of works during the whole of that period.

In 1864–65 the average monthly discharge for the year was 1784 cubic feet per second; in the Kharif season, 1791; and in the Rabbi season, 1777 cubic feet per second.

In this year the value of the irrigated crops being fifty times the water rent paid, it was resolved to increase the water rates, and this was actually done in 1867–68;—in this latter year the rainfall was exceptionally favourable to the cultivator, the result being that only two-thirds the breadth of wheat of the preceding year was irrigated; but as there was an increase of irrigation of 7436 acres of sugar-cane, the loss was made up.

The acreage of the principal irrigated crops on this canal for several years was as follows:—

	1860-61.	1861-62.	1862-63.	1863-64.
Sugar-cane, annual ...	26 102	33 782	44 730	30 089
Rice ... kharif	44 965	58 578	57 925	47 353
Cotton ... kharif	43 706	33 558	25 549	45 882
Wheat, rabbi	181 208	148 317	111 129	145 234

	1864–65.	1865–66.	1866–67.	1867–68.
Sugar-cane, annual ...	29 786	34 028	19 773	27 209
Rice ... kharif	57 157	51 517	62 071	39 455
Cotton ... kharif	77 738	62 684	104 796	98 800
Indigo ... kharif	1 131	1 477	1 805	1 315
Wheat, rabbi	163 159	126 293	150 233	100 937

In 1871, Col. Crofton proposed, with an estimate of £214 267, to make a permanent head, to complete the drainage works and the distributaries from Indri to Delhi and Jhind; it had however been discovered, in 1867, that the swamps near Karnal and on the Delhi and Rohtak branches were absolutely necessary;—the former having existed for 25 years, in consequence of the canal from Baria to Karnal consisting principally of natural channels.

The present state of this canal as regards works, financial condition, and irrigation, is shown in the tabular statistics.

The Eastern Jamna Canal is generally very similar in character to the Western Jamna Canal;—it was constructed in about the same time and the same manner, being an old, fully developed, and very remunerative perennial canal: its cost was about two-thirds, and its average irrigated acreage about one-half of that of the latter. It is also a restoration and enlargement of an old native work, commenced by the British in 1823.

The Eastern Jamna canal takes its supply from the Jamna at Kharrah, and passes it down the old bed of the Jamna for four miles, to Nayashahr, where is the regulating dam with 30 sluices and head of the main canal. In the first ten miles it crosses the mountain drainage at right angles, having dams at each of the torrents, and then continues on the high land of the country, on the watershed between the Hindan and the Jamna. The canal is in embankment for 40 miles, its water level being from 6 to 12 feet above the level of the country. The canal system now consists of 130 miles of channel and 625 of distributaries, watering a tract 120 miles by 15.

In 1830, water was admitted through its main canal, after an expenditure on works of £31 124; in 1837, the capital account had increased to £46 000, and in that year, which was one of famine, it yielded £10 084 in water rate, and about the same amount in increased land revenue, or in all about £20 000 or 44 per cent.; the acreage then being only 96 000; the value of crops saved by irrigation was £188 494, or eleven times the cost of the canal. In 1846–47, the capital account was £81 460, and the acreage was 106 705, yielding £12 175 as water rate, and £14 965 as increased land revenue, or as gross returns 25 per cent. on the capital. The works completed up to that time were as follows:—Channels main and branch, 465 miles; irrigation outlets 136; dams, 11; drainage outlets, 1; aqueducts, 7; bridges, 71; inlets and escapes, 26; falls, 14; mills, 12; workshops and station houses, 43.

As to the amount of irrigation effected by this canal in its earlier stages of developement, comparatively little is known; in 1832–33 the tract irrigated was 276 square miles, yielding £248 177 in water rate, and £136 742 increased land in revenue; while in 1850–51, the irrigated tract was 497 square miles, yielding £384 919 in water rate, and the same amount of increased land revenue as in 1832. A portion of the canal was remodelled in 1854, and new escapes were made, which have since formed injurious swamps: in fact, even at present, the necessary drainage works can hardly be said to have been fairly

taken in hand. From the year 1863–64 the water rates were enhanced, and the repairs to distributaries carried out by Government, and charged to maintenance; certain improvements were also effected by drainage works. At this period, a large amount of water was usually sold by contract, 288 villages taking it in that manner.

The acreage of the principal irrigated crops grown, of which the wheat, barley, and indigo form the greatest portion of the Rabbi, or cold weather crop, was as follows for four years:—

		1864-65.	1865-66.	1866-67.	1867-68.
Sugar-cane, annual		28 530	29 034	20 847	26 987
Rice ...	kharif	28 020	39 091	37 122	41 345
Cotton ...	kharif	14 405	2 887	5 080	2 646
Wheat and barley...		79 490	74 327	139 257	96 489

In 1871–72, the gross returns amounted to nearly 30 per cent. on the capital. The data of the works, the finance, and the irrigation for recent years will be found in the tabular statistics.

The Ganges Canal, commenced in 1848, and opened in 1854, is the third of the large perennial canals of Northern India, made by the British. It may be considered at present to be like the Bari Doab, a half-developed canal, in contradistinction to the Eastern and Western Jamna canals, which have their irrigation fully developed. As it seems to be the fate of so many Indian canals to be allowed to remain in a partially developed condition for a long time, their results when in this stage are naturally interesting, although they do not admit of close comparison with those of completed canals.

The principal head of the Ganges canal is about 2½ miles above the sacred town of pilgrimage, Hardwar, or Haridwar. In the first 18 miles of its course the canal passes the Ratmu, the Ranipur and the Pattri torrents, the former torrent passing through at the same level, and the two latter in masonry superpassages over the canal. At the 18th mile, above Rurkhi, the canal crosses the Solani river in a masonry aqueduct; the embankments of approach are about 30 feet above the valley, and are 3 miles long; the aqueduct itself is 920 feet long, in fifteen arches of 50 feet span, and 30 feet in height. From this point onwards the main canal nearly follows the watershed between the Ganges and the Jamna for about 181 miles to Nanun, throwing off branches and cuts for irrigation and navigation. From Nanun the eastern branch, 170 miles long, continues to Etawah, where it falls into the Jamna, and the western branch of the same length continues to

Khanpur, where it falls into the Ganges. There are also two smaller branches, 83 and 10 miles long respectively. This canal is of immense size; it carried a supply of 5100 cubic feet per second in 1870, and utilized 90 per cent. of it; besides this it has an irrigating capacity of 1 205 000 acres. As to dimensions, the first four miles from Haridwar are in natural channel, a branch of the Ganges. From Mayapur, where the artificial canal begins, and for a distance of 50 miles, the canal has a constant bottom width of 140 feet, a depth of 10 feet, and a slope of bed of 1·5 feet per mile. From the 50th mile where the Fattahgarh branch takes off, down to the 110th mile, where the Bulandshahr branch takes off, the bottom breadth is 130 feet, and the depth 9 feet: from the offtake of the Bulandshahr branch to that of the proposed Koel branch, the bottom width is 110 feet, and the depth 8 feet; thence to Nanun the depth remains the same, but the bottom breadth varies from 96 to 80 feet. The Fattahgarh branch is at present 83 miles long, the Bulandshahr branch 54 miles long; the Khanpur and Etawah branches are each 80 feet in bottom width at their heads, diminishing gradually to 20 feet at their lower extremities.

Of the details of the works as originally contemplated, there is ample given in the large work of Colonel Sir Proby Cantley, the designer and constructor of this canal, of whose energy, patience, and perseverance, it is impossible to speak too highly, when reflecting on the difficulties, both political as well as other, that he had to encounter.

In spite, however, of the large amount of money and energy spent upon this canal, it is a particularly unfortunate one. Its works were once stopped for some time, owing to the caprice of a Governor-General, who wished it to be made into a purely navigation canal; it was defective in several important respects, the inclination allowed to its bed was far too high, its bed retrogressed and its falls were damaged, so that it could not carry its full supply until about 1866, when a large additional outlay had been made. In fact, the whole of the canal, main and branches, had to be remodelled throughout; and the distributaries had been so badly laid out, that hundreds of miles of them have been abandoned at various times. The remodelling of the canal commenced in 1864, is now going on; and it is to be hoped that it will eventually carry the full supply originally intended, without increasing the capital account, now £2 605 178 to much beyond £3 000 000. While 4700 cubic feet per second is the highest amount of supply yet utilized on this canal, it is probable that eventually it may rise as high as 5500, the supply for which it was originally designed and intended being 6750 (or 7000?) cubic feet per second; should it, however, after

complete remodelling, arrive at that irrigating power, it will then have only six times the supply of the Eastern Jamna canal, at a cost of about twelve times as much that of the latter.

The acreage of the principal crops irrigated during four years was as follows :—

	1864 65.	1865-66.	1866-67.	1867-68.
Sugar-cane, annual	50 159	58 416	46 338	55 232
Rice } kharif	22 466	23 134	30.539	36 365
Cotton } kharif	42 026	10 496	19 094	5 616
Indigo	35 166	47 714	70 487	75 684
Wheat and barley	338 971	362 679	400.444	319 715

About two-thirds of the irrigation effected by this canal is flush, or delivered at the ground surface, the remainder is delivered at a low level, the water being raised to the surface by native mechanical contrivances. In order to carry out the irrigation of the whole of the tract intended, it is proposed to make a secondary headworks at Rajghat on the Ganges, and to supplement the Ganges canal by new works, named the Lower Ganges canal, estimated to cost £1 825 000 in addition; these works were commenced in 1872, and £54 439 spent in construction during that year.

Details of the expenditure on works, the returns, and the irrigation from this canal during late years, are given in the tabular statistics.

The Bari Doab Canal, from the Ravi in the Panjab, is the fourth of the large perennial canals of Northern India.

It is a half-developed canal, undergoing a process of remodelling, and hence very similar to the Ganges canal. It was commenced in 1850, with an original estimate of £530 000, and the greater portion of the main canal and works are now finished; as no account of the detail of progress is forthcoming, it will be best to describe the project as contemplated.

The canal is taken off from the left bank of the Ravi near Madhopur, and after a length of 28 miles throws out the Kasur branch at Tibari: at the 7th mile of the Kasur branch, the Subraon branch takes off; these two branches will be 90 and 67 miles long respectively, the former tailing into the Kasur nalla at Aljowan, the latter into the Tatti nallah at Subraon. The portion of the main canal from the head of the Kasur branch to that of the Lahor branch, which is situated in the 52nd mile near Aliwal, is designated the Upper main branch, and is 24 miles long. The remaining portion of the canal, from the head of the Lahor branch to the Vahn escape, into which the

canal tails, is called the Lower main branch, is 88 miles long, passes the town of Amritsar, and discharges itself through the Vahn escape into the Ravi. The Lahor branch from Aliwal passes Lahor, and tails into the Ravi at Niazbeg, 9 miles below Lahor: its length is 59½ miles.

The section of each branch is as follows:—

	Breadth at head.		Breadth at tail.		Depths.	
	Bottom.	Mean.	Bottom.	Mean.	Highest.	Lowest.
Main line	112	120	112	120	4·9	2·5
Upper main branch ...	84	92	80	88	5·6	2·8
Lower main branch ...	70	77	56	63	4·6	2·3
Lahor branch	50	55	38	43	3·3	1·6
Upper Kasur branch...	60	66	60	66	4·0	2·0
Lower Kasur branch...	46	51	20	25	3·0	1·5
Subraon branch ...	50	55	20	25	3·3	1·6

The highest depths given are those with the full supply of 3000 cubic feet per second, the lowest, those with the lowest recorded supply of 1000: the mean width is that of the wetted section at full supply.

The mean velocity, with a full supply depth of 4·9 feet, is 5·3 feet per second, and that with an average depth of 4·2 feet at the canal head is 4 feet per second.

The canal is capable of irrigating 654 000 acres with full supply at a duty of 218 acres per cubic feet per second.

The distributaries and escapes are as follows:—

From	Number of distributaries.	Total length. Miles.	Escapes.	Length. Miles.
Main line	15	93	Malikpur ...	7
Upper main branch ...	10	75	Gulpur ...	9
Lower main branch ...	16	256	Sirkian ...	6
Lahor branch	23	291	Aliwal ...	11
Kasur branch ...	Not yet determined		Vahn ...	16
Subraon branch	Not yet determined		Naizbeg ...	1½

In the neighbourhood of Pathankot, there are two hill torrents, the Jennah and the Chakki, which with their branches cross the line of the canal, and had to be diverted.

In 1856 it was found that the cost of the canal would not be less than £1 350 000, and work was therefore concentrated on the first 55 miles down to the Lahor branch. In 1859 water was admitted, and it was then found that, as in the case of the Ganges canal, the declivity of bed allowed was too great, the consequence being extensive channelling out in the sandy tracts and deep holes below the

falls; it was also discovered that the minimum supply of the Ravi, calculated to be 2753, was actually only 1414 cubic feet per second, or less than the works were designed to carry.

In 1860, a native canal, the Hasli, yielding £84 985 by direct returns, and £86 387 by enhanced land tax, was incorporated in the account of the Bari Doab canal, which then yielded nothing.

In 1870, or eleven years after the above-mentioned discovery, the remodelling of the canal was commenced, and the Kasur and Subraon branches proceeded with, but as an additional supply from the Beas involved fresh works, the estimate of the canal and branches rose to £2 000 000. Progress in the remodelling was going on in 1872–73, and the headworks at Madhopur were nearly completed. At present the aggregate length of main canal completed is 212 out of 247 miles, and of distributaries, 692 miles. In spite therefore of everything to the contrary, the irrigation from this canal in 1872 brought in a gross return of £81 876, or a net return of £50 216, or nearly 4 per cent. on the capital.

The acreage of the principal irrigated crops grown during four years was as follows:—

		1864-65.	1865-66.	1866-67.	1867-68.
Sugar-cane, annual ...		9 878	9 181	9 156	10 600
Rice	kharif	29 212	53 564	57 615	63 691
Cotton ...	kharif	3 881	5 236	12 511	21 101
Cereals, rabbi		97 722	59 827	108 707	122 720

The estimated value of the irrigated crops grown is as follows, for several years:—

In 1860–61, £256 024; in 1861–62, £307 238; in 1862–63, £192 668; in 1863–64, £241 969; and in 1872–73, £913,706.

Details of the works, the finance, and the irrigation from this canal are given in the tabular statistics.

The Minor Canals of the North-West Provinces.

The Dun Canals consist of five perennial canals of an aggregate length of 66 miles in the Dera Dun, a valley of the Sawalikh, or lower Himalayas, north-west of Hardwar:—they consist of—

	Miles long.	Discharge in 1872-73. C. ft. p. sec.	Supply utilized. C. ft. p. sec.	Opened in	Acreage in 1872-73. Acres.
The Bejapur ...	11	39	30	1840	5432
The Rajpur ...	12	11	9	1843	2736
The Kuttapatthar	19	33	17	1854	28
The Kattanga ...	13	25	15	1859	20
The Jakhan ...	12	15	9	1863	[illegible]

The acreage of irrigated land was not fully measured until 1867.

The distributaries have an aggregate length of 67 miles. At that time the capital outlay amounted to £54 365; the direct and indirect returns for that year were £3518 and £475, of which £1862 was mill rent, while the working expenses were £2514; in 1872–73 the capital expended was £57 253, the direct and indirect returns for the year £4791, of which £2390 was mill rent, and £475, and the working expenses, £2504; the acreage irrigated in each of these years being thus:—

	Kharif.	Rabbi.	Total.
1867–68	4334	7654	11 988
1872–73	5217	8785	14 002

The water rates were reduced in 1871, thus causing a temporary loss; but in the future these canals will, after the improvements now in progress are effected, yield higher returns.

The Rohilkand and Bijnaur Canals.—These consist of a number of ancient badly-designed lines, which are worked at a loss at present, though after remodelling may yield very good result:—they are

The Baigrul group	...	...	108 miles.
The Kitcha Dhora group		...	32 „
The Paha	„	...	13 „
The Kailas	„	...	32 „
The Nagina / The Nehtor	Bijnaur		38 „

The capital outlay up to 1872–73 was £103 600; the direct, indirect, revenue and working expenses for the year £3438, £2261, and £5132 respectively; the acreage, Kharif 21 204; Rabbi 34 446; Total 55 650 acres. The length of distributaries was increased from 180 miles in 1867–68 to 294 miles in 1872–73.

The Sarhind Canal, from the Satlaj in the Panjab, is a perennial canal now under construction. It was originally projected by Sir William Baker, in 1840, the detailed project was submitted by Colonel Crofton, in 1862, and estimates for the works to the value of £2 980 427 were sanctioned early in 1872.

The headworks are at Rupar, a town at the foot of the hills. At the 38th mile (these are canal miles of 5000 feet) the main canal crosses the Grand Trunk Road, and the railway from Ludhiana to Ambala. At the 41st mile the main canal ends, and the feeder line and the combined British branches take off. The length of the combined British branches is to be 3 miles, after which they will divide

into the Ubohar branch, 125 miles long, and the Bhatinda branch 100 miles long; the former of these will be navigable up to its 51st mile, whence the Satlaj navigation channel will take off and after a course of 45 miles tail into the Satlaj. The feeder line, which is a continuation of the main line, will be divided into three sections by the heads of the Kotla, Gaggar, and Choa branches of the canal, belonging to native states, which take off on the right side of the line; the lengths of the three sections of the feeder line being 14, 16, and 9 miles respectively, while that of the three branches are to be 90, 56, and 25 miles. The end of the feeder line is to be the point of junction of the heads of the Choa branch and the Patiala navigation branch. The latter will be 6 miles long, and will tail into the Patiala nallah near Patiala. The Choa branch will for the present tail into the Gaggar river, although it was proposed to connect it with the Western Jamna canal by a navigation cut 55 miles long, joining it at Indri.

This canal being partly for the benefit of native territory, one-third of its cost will be borne by three native states.

Up to the end of 1870–71, the capital account amounted to £185 667, of which half was expended in works; to the end of 1871–72, £415 186, of which £276 260 was on works; to the end of 1872–73, £501 315, of which £425 078 was expended in works, independently of establishment; of the latter sum, £240 613 was expended on about 200 million cubic feet of earthwork, and £107 010 on head and regulating works.

This canal with its branches will be 554 miles long, and will irrigate 783 000 acres in a most neglected tract of country.

The Agra Canal is like the Sarhind canal, a perennial canal under construction; it will irrigate a tract on the right bank of the Jamna, between it and the Khari Naddi, from below Delhi to the Utangan river below Agra.

The total length of main canal is to be 140 miles, its bottom width at the head, 70 feet; its supply will be 1100 cubic feet per second in the Rabbi season, and 2000 cubic feet per second in the Kharif season, requiring respective depths of 7 and 10 feet. The irrigable area is about 1200 square miles, of which about one-tenth is unculturable waste, and one-fifth is irrigated from wells.

The supply of the Jamna at Okhla having lately been found to fall occasionally below 800 cubic feet per second, in May 1870 having been only 472 and in January 1871 only 756 cubic feet per second, the supply

of the Hindan, which is capable of giving 300 cubic feet, will also be used in supplementing the canal, giving altogether 800 cubic feet as a certain minimum supply, according to which the depths needful for navigation are determined.

The fall of the canal from the head to the 32nd mile is ·5 feet per mile; at this point is an overfall of 5·75 feet, and beyond that to the 86th mile, the gradient is 1·0 per mile; after which it varies from ·33 to 1·00 feet per mile; below the 117th mile it becomes a simple distributary.

The discharges and velocities intended are as follows:—

Mileage.	Bottom width.	Depths.	Mean Velocities.	Discharges.
Head to 32 ...	70	5·8 10·6	1·82 2·36	800 min. 2000 max.
32 to 40 ...	58·8	4·1 7·0	2·25 2·76	587 „ 1262 „
40 to 50 ...	53·4	4·3 7·2	2·29 2·88	574 „ 1239 „
50 to 60 ...	47·4	4·1 6·9	2·28 2·82	485 „ 1044 „
60 to 70 ...	41·4	4·1 6·8	2·27 2·75	429 „ 910 „
70 to 80 ...	30	4·2 6·8	2·26 2·69	326 „ 670 „
80 to 85⅜ ...	24·2	4·4 6·6	2·20 2·62	276 „ 535 „
85⅜ to 95⅝ ...	24·2	4·9 7·0	1·24 1·41	176 „ 309 „
95⅝ to 100 ...	24·2	4·8 7·0	1·22 1·41	172 „ 303 „

From 100 to 117 miles the bottom widths vary from 21 to 18 feet; the depths from 3·7 to 5·2, the velocities from 1·5 to 2·3, and the discharge at the 117th mile is from 130 to 203 cubic feet per second.

The headworks at Okhla were begun at the end of 1868, and generally open in 1873; the supplementary headworks on the Hindan, below the Railway Bridge, are connected with the former by a canal having a bottom width of 24 feet, and discharging 291 cubic feet per second with a depth of 5·6 feet; it is 9 miles long, and enters the Jamna at one mile above Okhla, where there is a lock to prevent the return of flood water. The distributaries have discharges varying from 140 to 25 cubic feet per second; the principal works, bridges, escapes, and weirs are comparatively inexpensive. The total estimated cost of the Agra canal is £540 788, of which £124 200 is that of

headworks; the total area of irrigation is calculated at 704 000 acres, and the probable net income when the irrigation is fully developed is expected to be £51 375, in addition to £4000 from navigation and mill rent—or about 10 per cent. net.

Up to the end of 1872-73, the capital account stood at £432 267, of which £302 692 was incurred on account of works and plant, and £73 183 on establishment, this amount having been spent in five years. Of the above outlay, £30 131 was spent on plant, £106 444 on earthwork, £80 014 on falls and weirs, £37 736 on bridges, and £11 522 on buildings, and the remainder on miscellaneous works.

The Orissa Canals.

A series of main canals in the Orissa delta for navigation and irrigation, with headworks and distributaries.

The headworks proposed for these canals consist of three weirs across the Mahanaddi, the Katjuri and the Beropa, 6400, 3900, and 1980 feet long respectively; the two first 12·5, and the third 9 feet high; they are of modern design, having movable iron stanchions and shutters, that admit of being lowered to allow floods to pass over them. The canal for the irrigation of the central delta, between the Mahanaddi and the Katjuri, is taken off from the right flank of the Mahanaddi weir, and a junction canal will connect it with the Katjuri. The Taldandah canal also takes off from the right flank, and runs to Taldandah, the limit to tidal navigation, and it, with its branch, the Machgong canal, will eventually irrigate 155 000 acres of the central delta; they can now irrigate 30 000, being in use for about one-third of their lengths, or 52 miles of each. Two canals are led off from the Beropa weir: the one from the left bank is the high-level canal, designed for navigation from Kattak to Calcutta; of this the first 32 miles to the river Brahmani are open, and the greater part of its distributaries for the irrigation of 80 000 acres are completed: the other from the right flank of the Beropa weir, intended to irrigate the country between the Mahanaddi and the Brahmani, is called the Kendrapara canal; it is 160 feet wide and 7 feet deep, and is intended to irrigate 270 000 acres of the northern delta, at a duty of 120 acres per cubic foot per second of supply; the distributaries have an aggregate length of 171 miles, and will irrigate 85 000 acres; and its Pattamandi branch taking off on the fourth mile, and running to a port on the estuary of the Brahmani, will irrigate 113 000 acres.

The present estimate of the cost of these works is £2 598 200, and they are intended to irrigate 1 600 000 acres.

The Midnapur canal, opened in 1871, connects Midnapur with tide water in the Hughli, 16 miles below Calcutta, and forms a communication between that river and the Kusi, Rupnarain,-and Damuda. It will be 52 miles long, and will effect the irrigation and drainage of 200 000 acres: it is now capable of irrigating 72 000, but its distributaries and drainage channels are still incomplete. Its estimated cost is £931 000.

The history of the Orissa Canals is as follows:—

The preliminary designs, drawn up by Col. Sir Arthur Cotton, in May, 1858, were estimated to cost £1 300 000, and intended to irrigate 2 250 000 acres. A charter was granted to the E. I. Irrigation Company in June, 1861, and capital was raised to the amount of one million as a first issue. Surveys, preliminary designs, and estimates were drawn up afterwards under Col. Rundall by May, 1863; the estimate amounting to two millions, and the proposed amount of irrigation one and a half million acres, at a duty of one cubic yard per hour per acre.

Certain initiatory works were estimated in detail, thus:—

1. Head works, comprising the Naraj weir, the Mahanaddi anicut, the Beropa anicut, and the Kattak head-works, 1500′ long × 7½′ high	£165 996
2. First Section of High-level Canal, 32 miles from the Mahanaddi to the Brahmani ...	58 449
Its distributaries, 112 miles for 87 000 acres ...	13 050
3. Kendrapara Canal, 40 miles, Kattak to False Point ...	33 537
Its distributaries, 180 miles for 270 000 acres ...	40 500
4. Midnapur Canal, 48½ miles, Midnapur to the Hughli ...	152 342
Its distributaries, 160 miles for 148 500 acres ...	22 275
5. Tidal Canal, first two reaches 27 miles from the Rupnarain	49 119
	535 268
30 per cent. for stores and management	160 580
	695 848
Surveys of general scheme, purchase of a fleet of boats, London Offices, and preliminary expenses had already cost	123 935
Interest already paid to shareholders	112 477
Total estimated cost of initiatory scheme ...	£932 260

Estimated return. Navigation to repay establishment and management, and the irrigation of 505 500 acres, at 5 Rs. per annum, to yield a gross return of 36 per cent. on the £695 848, and deducting 5 per cent. for repairs and maintenance, 31 per cent. net; or 21 per cent. on the million of total expenditure estimated.

The works were begun in December, 1863. Irrigation was first available in December, 1865, was first taken up in April, 1866, and began to yield returns in October, 1866. Navigation began to yield returns in March, 1865. The Company sold the Orissa undertaking in December, 1867; the works constructed and returns being as follows:—

The total amount of work done by 31st May, 1867, under the heads of the preceding estimate, was—1. Headworks open, but not complete; 2. High-level Canal, 10 miles open, 12 nearly ready, and 17 miles of distributaries open; 3. Kendrapara Canal, 30 miles open, to a reduced width, and 72 miles of distributaries open; 4 Midnapur Canal, 28½ miles under construction, 10 nearly ready, and 46 miles of distributaries open; 5. Tidal Canal, 27 miles open without locks. Water was then available for 153 400 acres of irrigation.

Between May and December, 1867, further work was done on the above canals, details of which are wanting, as well as 23 miles of uncompleted work on the Taldandah canal. The expenditure, up to October, 1867, was as follows:—

Expenditure—on works up to June, 1867	...	£620 000
,, from June to October	...	187 936
,, from Oct. to Dec. 1867	...	not known.
Total expended on works in India ...	...	807 936
Total on all accounts	...	884 861
Balances	...	58 671
Receipts—not including Govt. loan of £120 000	...	£943 532

Returns from irrigation in October, 1866, and February, 1867.

At 5 Rs. ...	1067 acres and	573 acres ...	£ 821
At 3 Rs. ...	1018 acres and	2572 acres ...	1077
At 1½ and 1 Rs.	261 acres and	1183 acres ...	188
		Total, 6674 acres irrigated ...	£2086

At this time water was available for 60 000 acres.

Returns from irrigation at the end of October, 1867.

3007 acres, at 5 Rs.	...	...	£1504
1045 " " 3	...	...	313
385 " " 2	...	...	77
1799 " " 1	...	...	180
3600 " " $\frac{1}{2}$	...	...	180
Total 9836 acres	...	...	£2253
Water for 13 000 acres stolen, value	...	...	£2500

At this time water was available for 153 000 acres.

Returns from navigation, beginning March, 1863.

During 1863, £376; 1864, £843; 1865, £1089; 1866, £1445; in 1867, to 31st August, £1669. Total Navigation Returns 5922

Total returns, of which £3500 was not realized ... £13 761

At the time of sale, the Company had water available for 200 000 acres, which at 5 Rs. per acre would yield £100 000, or about 10 per cent. on the total expenditure, had the cultivators taken the water; as however they did not, and the Act had not then been issued (passed in February, 1870) to recover rates from land brought under water-command, it would have been unwise to extend the works, and the Company were then forced to sell up at par to the Government.

From 1867 to 1873, these works have been carried on by the Public Works Department. On the 1st April, 1873, the capital accounts stood thus:—

The Mahanaddi Project, including the Brahmani and Baitarni Series	£1 221 577
The Midnapur Project, including the Tidal Canal ...	695 812
Total	£1 917 389

The state of the works was thus in 1872–73:—

	Miles of canal completed.	Miles of distributary open per acre commanded.	Area for which water was provided.	Area for which distributaries had been constructed.	Cost per mile on original construction of canal	Cost per mile on original construction of distributaries.
			Acres.	Areas.	£	£
High-level Canal...	37	·0021	74600	42660	3618	203
Kendrapara Canal	40	·0032	313000	100000	2116	129
Taldanda Canal ...	27½	0042	155000	15336	1398	109
Machgong Canal...	6	·0040		16829	716	95
Midnapur Canal ...	24	...	138150	69950		
Tidal Canal.........	62	...	11500	2000		

The expenditure mentioned does not include establishment nor proportionate cost of headworks.—The supply provided for the areas was at the irrigating duty of one cubic foot per second for 133 acres.

The discharge passing down the Kendrapara canal varied from 500 cubic feet per second in August, to 126 in March, and in the high-level canal from 350 in July, to 115 in March; each of the canals were closed for repair for about two months in the cold weather.

In 1869, the water rates having been lowered from 10s. to 2s. per acre, the gross revenue amounted only to £141; in 1869–70, it amounted to £5235; in 1870–71, the acreage actually irrigated was 22 128 acres; and in 1871–72 only 11 652 acres, demands for water rate being abandoned by the revenue collectors, and only £1772 being actually collected.

In the year 1872–73 the total acreage of irrigation was only 4753 acres, yielding £4263 in water rate, and the navigation returns on a tonnage of 154 422 tons amounted to £4750; the total receipts, including £1481 from various other sources, amounting to £10 293, the highest year's revenue yet obtained.

The Son Canals.—These constituting a portion of the Bahar project of Colonel Dickens, are designed to provide high-level navigation for 295 miles from Mirzapur on the Ganges through Dehri, the headworks on the Son, to Manghir on the Ganges, and to irrigate the country on both banks of the Son, between this line of navigation and the Ganges. The Western main canal, from Dehri to Mirzapur, will be 125 miles long, and will command the irrigation of an area of 2100 square miles; the Eastern main canal from Dehri to Manghir, 170 miles long, commanding 3000 square miles. The main canals are designed to carry 5300 cubic feet per second, with a depth of water of 9 feet, and a bottom width of 180 feet; in the Eastern canal the fall from the Son to the Ganges, of 123 feet, will be overcome by a series of locks. It was originally intended that these and other works should have been carried out with English capital, under the East-India Irrigation Company in 1867:—they were however commenced in 1870 by the Public Works Department, under Mr. Levinge, aided by about twenty English engineers.

The Western Main Canal was nearly completed to full dimensions for a length of 22 miles by the end of March, 1873; and its bridges and siphons were in progress. The Eastern Main Canal was then also nearly completed for eight miles. On the Arrah Canal, which is to be 70 miles long, and will irrigate 430 000 acres, ground had been broken over 60 miles; and six locks, two bridges, and seven siphons were in progress. On the Patna Canal, which will be 84 miles long, and will

irrigate 390 000 acres, two-thirds of the earthwork was executed in 1872-73.

At the headworks, the masonry well blocks of the upper breast-wall of the weir were sunk right across the river in 1870-71, and in 1871-72 those of the lower breast-wall, as well as parts of the head and under sluices and head locks; the stone being brought by locomotives from quarries seven miles off.

The following is an abstract of the estimate of cost of the works:—

295 miles of high-level main canal at per mile,	£4000	£1 180 000
240 miles of main irrigation and navigable canal, at	£3000	720 000
928 miles of main irrigation distributaries ... „	£500	464 000
261 000 acres irrigated in detail ... ... „	£2	522 000
326 250 acres of minor drainage works ... „	8s.	130 500
Headworks		225 000
Workshops, shelter, &c.		43 000
		3 284 500
Superintendence at 12·5 per cent.... ...		410 500
Tools and plant		80 000
		£3 775 000

The capital account is as follows:—

	Works and Plant.	Establishment.	Total.
Up to 1st April, 1872	£368 036	£77 456	£445 493
During 1872-73	210 951	40 635	251 587
Up to 1st April, 1873	£578 987	£118 091	£697 079

The Son weir is 2½ miles long and 8 feet high, and is especially interesting as an example of the most modern construction, exhibiting like the weirs on the Orissa canals, also designed by civil engineers, a vast improvement over everything yet done in works of this class in India. It is probable that these canals will be partly open in 1875.

The Bandalkand Canals, from the rivers Betwa and Dassan proposed by the late Captain A. H. Bagge, of the Bengal Engineers, still remain as projects under contemplation: detailed surveys were, however commenced in 1873.

THE INUNDATION CANALS OF THE PANJAB.

1. *The Lower Satlaj and Chenab Canals.*—The canals from the Lower Satlaj are 19 in number, and have an aggregate length of 418 miles; those from the Chenab are 13 in number, and have an aggregate length of 222 miles;—the whole of these, excepting 19 miles, were constructed and in working order at the time of the British annexation:

the breadth of these canals varies from 5 to 36 feet, and their depth of water from 3 to 11 feet; they have no distributaries, irrigation being supplied direct from them by means of private water-courses.

2. *The Upper Satlaj Canals* are four in number:—

	Length.	Breadth.	Depth.	Distributaries.
1. The Katora ..	66 miles	33½ feet	3·5 feet	47 miles.
2. The Khanwah ...	81 „	60 „	6 „	
3. The Upper Sohag	57 „	40 „	4 „	
4. The Lower Sohag	20 „	20 „	3 „	

The first was constructed by the British Government, and opened in 1870. The second was constructed, for a length of 63 miles during the reign of Akbar: it was reopened in 1843, and extended by the British Government for 18 miles from Dewalpur southward; 25 miles of distributaries were also constructed at that time. The third was constructed by the British Government, and opened in 1855; it has two distributaries belonging to Government, 12 miles in aggregate length, and two to landholders of 16 miles, or 28 miles in all; a new head was completed in 1871 to serve as an alternative entrance to this canal, for occasions when the river sets in on the old head. The fourth was constructed by a landholder shortly after the British annexation. There is also another canal, called the Nikki, about which particulars are wanting.

3. *The Jhelam Canals.*—There are 18 inundation canals from this river in the Shahpur district; they were purchased from local funds in 1870. The dimensions of two of them are as follows:—

	Length.	Mean breadth.	Average depth.
Shahpur Canal ...	17 miles	18 feet	6 feet.
Sahiwal Canal ...	19 „	10 „	4·5 „

4. *The Indus Canals* are 13 in number, and have an aggregate length of 577 miles, varying from 9 to 97 miles in length; they are all drawn from the right bank of the Indus in the Dera-Ghazi Khan district, at the south-western corner of the Panjab frontier: their breadth varies from 11 to 60 feet, and their depth of water from 3 to 6·5 feet; they have branches, but none of them have separate distributary channels. They were all, except one of 67 miles, the Dhundi, running at the date of British annexation; but branches to the aggregate length of 32 miles have been added since, half the expense being borne by the British Government, and half by the proprietors of the estates benefited. In addition to the above, two canals, the Fazilwah,

and the Masuwah, have been constructed and maintained by private enterprise.

There are also some canals in the districts of Muzaffargarh, Peshawar, and Bannu, about which no information exists in the records.

In addition to the canals, there are a number of embankments, aggregating a length of 38 miles, in the neighbourhood of Dera-Ghazi Khan, that were constructed in 1854 and 1863 for the purpose of shutting out overflows in the rainy season, which used annually to devastate large tracts of country, and necessitate remissions of Government land-revenue.

The financial results and acreage of the Panjab Inundation Canals in 1872-73, were as follows:—

	Capital outlay up to end of 1872-73.	Returns of 1872-73.			Acreage irrigated in 1872-73.		
		Direct.	Indirect.	Working expenses	Kharif.	Rabbi.	Total.
	£	£	£	£			
Lower Satlaj and Chenab	10 520	12 938	21 330	16 362	149 143	93 361	242 504
Upper Satlaj	44 292	6 459	2 791	15 621	74 914	60 446	135 360
Indus	43 736	?	8 094	18 046	132 818	47 319	180 137
(average)	...	2 700	...	...	...	...	...
Jhelam	2 122	710	...	434	unknown.	4 445	10 513

Of the acreage irrigated by the Lower Satlaj and Chenab Canals, 20 per cent. was lift irrigation. The mean discharge of the Upper Satlaj Canals was 1742, and that of the Indus Canals 4107 cubic feet per second in 1872. The Jhelam Canals are under the management of the collectors.

The Canals of the Bombay Presidency.

The Sakkar and Shahdadpur perennial canal, from the Indus in Sind, commenced in 1861 with an estimate of £72 982, was opened in 1870; it is 63 miles long, will irrigate 140 000 Sindian bigas of land, and is expected to yield a revenue of £210 000.

The Sind Inundation Canals are of native origin, their names and lengths are as follows:—

West of the Indus.	Head.	Length in Miles.	
The Sind ...	21 miles below Sakkar ...	66	3 branches.
The Ghar ...	23 miles below Sakkar ...		2 branches.
The Western Nara	27 miles below Sakkar ...	70	300 ft. wide
The Bigari ...	unknown ...	48	40 ft. wide

East of the Indus.

The Eastern Nara, Rori, improved in 1859.			Acres.
The Mitrau branch of the E. Nara, British, 190 miles, irrigates			157 000
The Thar branch of the E. Nara	...	„	38 000
The Fullali	...	Natural branch of Indus irrigating Haidarabad.	

It is very doubtful whether a large proportion of these canals are not improved natural channels; there is very little information about the irrigation effected by them; they will probably be made eventually to serve as distributaries to perennial canals, having their heads at Sakkar, at Jhirk, 250 miles below it, and at Kotri.

The Jamda Canal, in Kandeish, was commenced with an estimate of £10 000, and was opened in 1869.

The Krishna Canal has its headworks at Karwar, in Sattara, its estimate was £58 133; in 1872, 32 miles of canal were finished, and 2038 acres irrigated, yielding a revenue of £955.

The Ahmadnagar Canal, estimated to cost £21 941 was opened before 1870.

The above comprises the whole of the canals of the Bombay Presidency. Information about them is very scarce.

THE CANALS OF THE MADRAS PRESIDENCY.

The Tumbaddra Canals.—The principal headworks of these canals consist of a weir across the rocky bed of the Tumbaddra at Sunkesala, 4500 feet in length of clear overfall; the section varies, but is everywhere 8 feet broad at the top, the alternate stones of the coping being 1 foot thick, 8 feet long, and weighing each 1¼ tons. The mortar used is Karnul kankar, except for the coping which is in Portland cement. The height varies from 6 to 26, averaging 18 feet; and the highest registered flood rose 7½ feet over the crest.

The main features of the canal are as follows:—the first 75 miles are designed to carry 3000 cubic feet per second at the head, and, after parting with one-fourth of this for irrigation, to convey the remainder through the Metakandal watershed cutting at its other extremity. There 1912·5 cubic feet per second can be discharged into the Kali, and 337·5 carried down the continuation of the canal. Of the 1912·5, 750 are taken up at a fresh offtake at Jatur, and 375 at Rajoli, leaving 750 for irrigation below Kaddapa.

The minimum section of the canal in the first 75 miles has a 90-feet bottom-breadth, with 2 to 1 side slopes. For the first 45 miles, the

n 2

fall is adapted to a maximum depth of water of 8 feet, below the 45th to one of 9 feet. The gradient of the canal is generally from ·3 to ·5 feet per mile, but in one or two deep cuttings 1·5 feet. Below the 75th mile, the natural watercourses of the Kali and the Kunder become the main channel of supply. The 1st branch channel forms the canal from the 75th to the 95th mile; it has a head sluice and lock at Lockinsula, from which it is an irrigating channel 6 feet deep for the first 6 miles, with a flow of 337·5 cubic feet per second. Below that it is a still water canal, of a minimum depth of 5 feet, and a bottom breadth of 45 feet, having a fall of 180 feet, overcome by 7 double and 5 single locks, of chambers 120 × 20; the greatest fall of a double lock being 21, and of a single one, 13 feet. The 2nd branch channel forms the canal, from the Jutur weir at the 95th mile, to the 146th mile; it is adapted for a depth of 6 feet of water down to the 1st drop lock at the 118th mile. The weir is 6 feet broad at the top, on foundations of shale; it has head sluices, scouring sluices, and an entrance lock, with a water cushion below the fall. Irrigation ceases at the 130th mile. From the 118th to the 146th mile the canal consists of level reaches with 5 feet depth of water, having 17 locks to overcome a fall of 188 feet, the maximum fall in any single lock being 14 feet. The bottom breadth throughout is 50 feet. The 3rd branch channel, from the Rajoli weir at the 146¼th to the 180th mile, has also a bottom breadth of 50 feet, and with 5 feet of water will discharge 375 cubic feet per second. The Rajoli weir is made of limestone rubble, and built on rock; its top thickness is 5 feet, its front batters 1 in 2, and its lower face is vertical.

Across the Penner at Adanimayapilli are the headworks and offtake of the projected continuation of the canal to Nellor; the weir is mostly founded on wells in sand; 8 miles of this canal are open, and supply 37·5 cubic feet per second for irrigation.

The Hindri aqueduct, carrying the canal 90 feet broad, and 8 feet deep, at an elevation of 32 feet over the Hindri by fourteen 40-feet arches, is an important work. No modules are used on these canals. The ordinary hand sluices are of two sizes, one 5 feet broad, and of 3·75 feet lift, the other 1·5 feet wide, and 1 foot lift; each is worked by turning round a vertical screw that lifts a cross head, to which the cast-iron shutter hangs, each turn of the screw raising the shutter 1 inch and being easily worked in cast-iron grooves by one man against an average head of water of 6 feet.

The cost of the canal for the first 75 miles averaged £8000 a mile and for the rest of its course £2900 a mile.

This Tumbaddra project was first brought forward by Col. Haviland; it was carried out by the Madras Irrigation Company, having been commenced under the auspices of Lord Derby, and sanctioned in 1861, the estimate by Government officials amounting to one million sterling; the headworks were opened, and water admitted, in 1864: as the works could not be completed within the estimate, a loan of £600 000 was made to the company by the Goverment in 1866, under the condition that these works should be completed in July, 1871. They were completed by that date, 216 miles of canals and 377 miles of distributaries, commanding 91 567 acres, being opened. In 1872–73, the acreage commanded was 156 570 acres, being in excess of that necessary, when taken up, to repay the 5 per cent. interest, namely 130 000 acres. The actual acreage irrigated and returns up to the present time stand thus:—

In 1870-71	1 478	acres,	yielding	£897
„ 1871–72	9 980	„	„	3541
„ 1872–73	9 505	„	„	5020
„ 1873-74	19 791	„	„	8161

The small acreage in 1870–71 was due to the damage to the canal caused by unprecedented storms; and for which insufficient escape had been provided. In 1871 this was repaired, and the canal improved, and in 1872 water was again admitted throughout the whole length of the canal, to a depth of from 2 to 5 feet. In 1873–74 the canal carried 375 cubic feet per second, or 50 000 cubic yards per hour, having a depth of 4 feet of water throughout.

The eventual irrigating power of this series of canals is assumed to be limited to 250 300 acres of rice cultivation, at a duty of 2 cubic yards per hour per acre, in places where the waste water is lost, and of 1¼ where it is again taken up by the canal; this is, however, on the supposition that these canals remain dependent on the rainy season supplies of the Tumbaddra; should storage reservoirs be employed, as intended, to render the canals perennial, this acreage may be doubled.

The Godaveri Deltaic Works were commenced in 1847; the headworks consist of a long low dam at Dauleshwaram, the head of the delta, where the river is 6000 yards wide, from which channels are taken off for the irrigation of the eastern, central, and western portion of the delta. The irrigable portion of the delta is 2500 square miles, less 25 per cent. for waste land, or 1 200 000 acres. The water

available is 12 000 cubic feet per second in the flood season, during July, August, September, and October, and 3000 as a minimum during the rest of the year; the former will, at the duty of 40 acres to 1 cubic foot per second, irrigate 480 000 acres of rice, the latter, at the duty of 120 acres, irrigate 360 000 acres of sugar-cane; hence two-thirds of the delta, or 840 000 acres, may be irrigated when the works are completed; at present the total acreage irrigated is 264 717 acres, or less than one-third.

The dam consists of several portions of masonry work raised to a height of 12 feet above the river bed, broken by islands amounting in length to 4500 feet, and connected by earthern embankments. The Dauleshwaram portion is 4872 feet long, founded on wells 6′ in diameter, and sunk 6′; it is 19′ thick, consisting of a core of river sand faced by a curtain wall 7′ high, and 4′ feet thick at the base, and a masonry counter-arched fall 28′ broad and 4′ thick; the waste-board of cramped stone is 19′ broad and 4′ thick, the apron 80′ broad of massive stones; on both flanks are masonry wing-walls and revetments, on the left flank a lock, head-sluices to the channel, and under sluices for silt. The Ralli portion is 2862 feet long, but has a core of rough stone. The Maddur portion is 1548 feet long; and the Vegeshwaram portion 2584 feet long, having a lock and head-sluices. The earthen embankments are 7000 feet long, and the length of wing-walls 2500 feet. The effective height of the dam may be increased by 2½ feet by means of planks held in the grooves of cast-iron standards, ·5′ square and 10′ apart.

The irrigation of the eastern portion of the delta is provided for by 25 miles of main longitudinal channel, 4 miles of main transverse channel, 75 miles of main branches by Samulkotta and Coringa, and a series of smaller transverse channels, making on the whole, with intended extensions, 220 miles of main channels, from which the village watercourses will be supplied. The supply for this portion of the delta will be 4000 cubic feet per second, or enough for 160 000 acres of rice, which is three-fourths of the culturable area.

The irrigation of the central portion of the delta is provided for by the Ralli channel and its transverse lines, which amount to 90 miles in length, and other channels 70 miles more, in all 160 miles; one of the branches of the Ralli channel crosses a minor branch of the Western Godavari, in the Gannaram aqueduct, which carries 500 cubic feet per second, and irrigates with full supply 26 000 acres of rice out of a culturable tract of 60 000. If this system of channels *carried* at full supply 4000 cubic feet per second, they would be able

to irrigate 160 000 acres of rice and 120 000 of sugar-cane, or in all 280 000, or five-sevenths of the culturable area, 352 000 acres.

The irrigation of the western tract of the delta, is provided for by a main channel breaking off into a series, having an aggregate length of 154 miles, an extreme western channel going to the Colair lake with a corresponding net-work of channels will amount to 100 miles; these main channels, with others of various sorts, will in all amount to 460 miles for the western tract, and will be capable of eventually irrigating 280 000 acres out of a culturable tract of about 480 000.

The original estimate of Colonel Cotton for these works, in 1845, amounted to £120 000, and in 1849 this amount had been spent and the original works half completed; a new estimate for £240 000 was then adopted, and in 1853, £150 000 had been spent. It seems that in 1849 the irrigated acreage was 127 320, yielding £41 351 gross returns, and in 1864 was 202 111, yielding £123 187 gross income, the working expenses being about £26 390, and the net income £96 797, or about 20 per cent. on a capital outlay up to that time, of about £470 000.

The present financial state is shown in the tabular statement. Of the progress of the works, or of the development of irrigation there is not any satisfactory account forthcoming; it would appear, however, that a quarter of a century has been spent in carrying out only one-third of the intended irrigation in a district where the natives are exceedingly anxious to take up water, and that the accounts are still involved in obscurity.

The Kistna Deltaic Works, designed by Captain Orr, were begun in 1854. The anicut at the head of the delta at Bezwara is 3750 feet long, 305 feet broad, and has a height 21 feet above the bed of the river, or 21 feet above dry season level of the water; it has under sluices on the flanks. At this point the river is 5 to 6 feet deep in the dry season, and 30 to 40 feet in the mansun season; the average flood is 24, and the highest 31 feet above ordinary low water. In the dry weather, from November to June, the supply of the river is so small, being principally due to springs in the bed, that the dry-season irrigation would be unimportant; in the rice season the stream is continuous, and is 20 feet deep. The irrigable deltaic area on each bank is 1 250 000 acres, requiring 31 250 cubic feet per second; each channel head however provides only 3800 cubic feet per second in the

rice season, the channel having a breadth of 90 feet of waterway, 10 feet depth of water, and a fall of one foot per mile. The details of the channels are thus:—

	Length.	Supply.	Breadth of irrigation.	Acreage.
On the right, or Guntur Bank:—	Miles.	C. ft. pr. sec.	Miles.	
1st Western channel	50	1200	1¼	
2nd Central channel	30	720		
3rd Eastern channel	45	1850	4¼	63 200
On the left, or Masulipatam side:—				
1st channel	48	1500	3 to 4½	
2nd Drog channel	57	1000	2½	

It appears that there are, on the whole, with some others not mentioned, 290 miles of channel, and that the total supply during the rice season amounts to only 7000 cubic feet per second. The acreage irrigated in 1872–73 was about 170 000, out of an acreage of 280 000 possible with the full supply; we may hence assume that only three-fifths of the irrigation is now developed. The revenue in 1855 was £8800, and in 1863, £50 000; the present financial condition is given in the tabular statistics.

All records of progress of works, and of development of irrigation on these works, are entirely wanting. At present the channels are being enlarged and widened, in order to convey enough supply for the irrigation of 430 000 acres.

The Pennar Deltaic Works were commenced in 1849, and opened in 1855;—they consist of an anicut at the ferry at Nellor about 1560 feet long, and the main or Sarvaipalli channel from it, with distributaries irrigating the right tract of the delta; that on the left bank being high land is not irrigable. The supply of the Pennar being precarious, the Nellor and other tanks are utilized in keeping water in reserve and supplementing the channels. In 1857 the anicut was breached for 282 feet; and the repairs were not completed until 1861. The acreage irrigated in 1863 was 32 874; the acreage in 1872–73 is stated to be 169 073; but it is probable that this is a mistake, and includes irrigation not dependent on the anicut, more especially as the gross proceeds for the year amount only to £8954; see tabular financial results. It is now proposed to enlarge the channels, and further develop the irrigation.

The Palar Anicut and Works, in Chinglepat and North Arcot, seem to be in the same financial condition as the Pennar works; see tabular

financial results. There is no official record available for ascertaining definitely anything about the progress and irrigation of these works.

The Poini, Alliabad, Cheyar, and other anicuts in North Arcot have their financial results given in the table.

The Vellar and other anicuts in South Arcot yield the very high net profit of 53 per cent. on a capital outlay of £52 055, which probably does not include the whole cost of the works. There is no information about them available.

The Kalerun Deltaic Works are an improvement and enlargement of very ancient native works, made under the Telingi rajahs. The grand anicut of Seringham was in 1804, when Tanjor was ceded to the British, a solid mass of rough stones, 1080 feet long, 40′ to 60′ broad, and 15 to 18 feet high; this gave irrigation both along the Kalerun and the Kaveri, on the former 165 000 acres, on the latter 504 900 acres, or 669 900 in all, which must have utilized, at the duty of 40 acres of rice cultivation to 1 cubic foot per second, at least 16 747 cubic feet per second of supply, of which 12 622 were required for the Kaveri, and 4125 for the Kalerun irrigation. In point of fact, however, the total volume in December, 1833, was 16 875 cubic feet per second, of which only 9375 went along the Kaveri, and as much as 7500 along the Kalerun. To remedy this an anicut on the Kalerun was made between 1834 and 1836 by Col. Cotton; it was 2250 feet long, and 6′ thick, its height 5·3 to 7·3 feet, made of brick, capped with stone, the foundations 3′ deep, built on three lines of wells 6′ deep, and 6′ in external diameter; the apron 21′ broad, and 1′ thick, of stone in hydraulic cement; there were twenty-two sluices, each 2′ wide, by 3·5 high, to clear the bed of silt. In the year following its construction 240 feet of the dam were demolished, but were instantly repaired. In 1843 additional sluices were made, giving a total clear lineal waterway of 130 feet, but these produced little good; and it became evident that in remedying one evil, the works were causing another, the Kaveri was likely to suffer from excess of water in the same way as the Kalerun had previously.

In 1845, Col. Sim made a regulating masonry dam, 1950 feet long, across the head of the Kaveri, and lowered the Kalerun dam for a length of 700 feet by 2 feet, this put the regimen of the Kaveri and Kalerun perfectly under control. The Kaveri channel is now a channel of irrigation only, it is sub-divided into small channels, and its entire volume utilized; the Kalerun channel, besides giving irrigation, is the

main drainage channel of the delta. The irrigation from these works is the most completely developed possible, and the returns enormously profitable; the navigation, a matter of very inferior importance in such a country, on the contrary, suffers from the dams and the silt deposited above them;—in fact, a lock on the Kalerun dam had to be turned into a double sluice.

The Lower Kalerun dam was made in 1837, over the Kalerun, at 70 miles below Seringham, the head of the delta, with the following object. At that time the Upper Kalerun dam had forced so much water into the Kaveri, that the water in the Kalerun was much lowered, and a large amount of land was thrown out of water command; the object therefore was to raise the water in the Kalerun, and recover the command of it. The length of the Lower Kalerun dam is 1900 feet; its section consists of two rows of wells, 6 feet deep, having a sand core $3' \times 4'$ in the middle, arched over, with 4 feet of solid masonry above them for the foundations, and a body wall above 7½ feet high: when the water level reaches to the top of the anicut, the depth of water in front is 7½ feet; it has 23 under sluices, giving 69 lineal feet of waterway, and an apron in rear 24′ broad, and 3′ thick. The channel head above this dam takes off water for the irrigation of a district, eight miles below, in South Arcot: and hence, though the principal object of this lower dam was not attained by it, it has yet effected a useful purpose. In 1863 and 1864 three very serious breaches were made in this anicut, the water leaking through, and probably also, under the wells, which seem to have been carried to about half the depth necessary in such a situation, and were unprotected by any retaining wall or apron in front: it appears that in these cases the irregularity of the bed caused the current to impinge and concentrate its effects on the portions of dam that gave way.

The acreage irrigated has been materially increased, as well as saved from ruin by the former works: before 1836 it was 670 000; in 1850, 716 524; and in 1872–73, 748 673. The increase of produce effected by irrigation in these districts varies from one-fifth to one-eighth the gross produce of rice, or five to seven bushels of unhusked rice (padi) per acre. The Government revenue in which the water rate is merged is two-fifths the gross produce, and varies in value from nine shillings in Tanjor to twelve in Trichinopoly, and fourteen and sixpence in South Arcot, having an average over the whole of the districts of twelve shillings. The increase of annual revenue due to the works would, therefore, on 78 000 acres amount to about £47 000, while the Government returns for 1872–73 show a gross return of £110 243; see tabular statis-

tics. It is probable, therefore, that a large portion of this latter sum is, strictly speaking, due to the works of the Telingi rajahs, constructed before Col. Sim put the regimen of the rivers under control. If this is the case, the percentage of net profit due to the British works must be reduced from 128 to 51 per cent. on the assumed capital outlay of £81 014. With reference to this latter sum, it appears merely to include the cost of the three dams and headworks, and their reconstruction and alterations from 1836 to 1850; if, however, we place to the capital account the cost of channels and irrigation works dependent on those dams, which seems according to some accounts to amount to £91 874 on original works exclusive of repairs, this raises the capital account to £172 888, and lowers the net profit to the more reasonable percentage of 24.

Apart, however, from the matter of returns, both of finance, of irrigation, and of works, in which it is hoped the Madras Presidency is commencing a new era, it is an undoubted fact that the complete control and utilisation of so large a river as the Kaveri, at so early a date as 1846, within ten years after the original commencement of the restoration of the works, are results not known to be achieved on any other irrigation works in the world up to the present time. They reflect lasting honour on the names of Colonels Sim and Cotton.

The Anicuts of Madura.—The Suruli, the principal tributary of the Vaiga, joining it after a course of 36 miles from Gudalur, is entirely utilized in the irrigation of the Kambam valley; there are ten anicuts across it, with channels and tanks; the first is situated at half a mile from Gudalur, whence a canal on the left bank irrigates rice lands for 5½ miles, and eventually falls into the Kambam tank: the others irrigate a narrow strip of rice cultivation on each bank in the lower part of the Kambam valley. On the Vaiga itself are two masonry anicuts, the Perani and the Chitani, situated 22 and 18 miles respectively above the city of Madura, which are said to have been built by two favourite dancing girls, favourites of one of the Naik kings of Madura: the channels from them are in bad order. Below the Chitani there are no dams, the slope of the ground allowing channels to be taken off without the aid of anicuts. The supply of the Vaiga is so deficient in its lower parts, in the Ramnad, that any irrigation from it is only on a very small scale.

The supply of the river Gundu is very small, the local rainfall being only 18 inches yearly; on it, east of the town of Kamudi, 18 miles from the sea, is an anicut large dam, made of loosely built

stone; a channel from it takes its water to the Kallavi lake. On the river Vaipar are several stone anicuts, and on its tributaries are storage tanks; the amount of irrigation effected from these two latter rivers is unknown.

The Anicuts of the Tambrapurni.—There are seven anicuts on this river. The first is the Thalay anicut, just below the falls of Papanassam, it is renewed annually with stakes and brushwood; it has two channels, one 10 miles long on the north bank, and one 6 miles long on the south, each ending in a tank. The second is the Nathiani anicut, 6 miles below the former, it is a very ancient structure, consisting of large blocks of stone placed obliquely across the river, and is 468 feet long; only one channel flows from it, for 12 miles on the north bank, which irrigates 1119 acres, yielding a revenue of £1297. The third is the great Kannadien anicut, built of cut stone, it is 9 feet high, and has a top width of 6 feet; it has also a large rough apron varying from 35 to 160 feet in width; the anicut is divided into two pieces by a rocky island. A channel from it on the south side is 22 miles long, irrigates 9574 acres, and yields a revenue of £17 981; the Kannadien channel flows through the town of Serun-Mahadevi, 9 miles west of Tennevelli. The fourth is the Kodagan anicut, six miles below the last, it is 2287 feet long, of cut stone roughly put together; it has one channel from it on the north side 10 miles long, irrigating 5433 acres, and yielding £6106 of revenue. The fifth is the Palavur anicut, 2 miles east of the town of Serun-Mahadevi, it is 2532 feet long, its channel on the south side is 26 miles long, supplies 54 tanks, and terminates near Palamcotta, and irrigates 2865 acres, yielding £5468. At a mile and a half below the Palavur is the sixth or Sutamelli anicut, 2 miles east of the town of Serun-Mahadevi, divided by a rock into two portions, its channel on the north side is 14 miles long, supplying two distributaries, passing through the town of Tinnevelli, which irrigate 1806 acres, yielding £3299 of revenue.

The seventh anicut, 18 miles below the last, is the Murdur anicut, 27 miles from the sea; it is of horseshoe shape, 4028 feet long, and supplies a channel on either side; its escape weir is of beautifully cut stone work. Its channels run in and out of several large tanks, and irrigate 14 400 acres, yielding a revenue of £17 700. Below this anicut are four channels, irrigating 4280 acres, and yielding £4980 of revenue.

The total amount of irrigation effected by these native works is

39 578 acres, yielding £56 828; the repairs only cost $1\frac{1}{2}$ per cent. on the revenue.

The English anicut at Strivigantam, 12 miles below Murdur, will be 1380′ long, 6′ high, and $7\frac{1}{2}'$ broad, founded on wells; it will irrigate 15 000 acres on the north and 15 000 on the south bank, and supply Tuticorin with water; it was commenced in 1869, on an estimate of £83 160; in 1873 £76 878 had been spent on construction; it is, therefore, probably nearly completed now.

The estimated amount of water from this river that is utilized for irrigation is given in the brief account of the river Tambrapurni, page [26].

The Anicuts and Channels of Maisur.

General description of Works.—The ordinary stone dam or anicut in Maisur varies from 7 to 25 feet in height, it consists of a mass of dry rubble, faced with large stones, placed on a rocky site; the front casing of stones $3\frac{1}{2}' \times 1\frac{1}{2}' \times 1'$; the rear aprons of large stone blocks $9' \times 3\frac{1}{2}' \times 2'$, each stone projecting for one-third of its length beyond that above it, or about $2\frac{1}{2}'$; the interstices are filled with small rubble; these works are unstable and leaky, allowing all the summer discharge to escape, and only supplying the channels in season of flood, when again they are easily damaged and breached; the dams are curved and point up stream, having a length about double the width of the river, the crown is lower near the head-sluices to relieve the pressure against them in flood. The head sluices consist of rough stone uprights, 4 or 5 feet apart with stone caps over them; the openings being stopped with brushwood or earth filling; they are very inefficient during floods, which frequently enter uncontrolled and make breaches.

The channels are rough trenches generally following the undulation of the country, and very badly levelled and set out; the irrigation water is taken direct from them through cuts made in their banks, the escapes for surplus water are made in the same way; the channels suffer much from silt brought down by cross drainage, also from breaching by the same cause; although there are rough stone silt dams as well as solidly constructed outlets at low levels for holding up and scouring out the silt from the channels.

Results.—The financial results, as shown in the tabular statistics, appear meagre in the extreme; the causes being that not half the irrigated land is assessed, and that the irrigation water is surreptitiously taken. It appears that if all the irrigation were paid for, the tanks of the Maisur division would yield £56 900, and those of the

Hassan division £84 450 more than the revenue collected; or that, roughly for the whole province, £200 000 a year remains unrealised.

According to paragraph 14 of Major Pearse's letter of 14th March, 1866, two British officials, Major Montgomery and Colonel Clerk, after several attempts to induce the landholders to pay for the water, were obliged to give it up.

Works recently reconstructed.—The Maddur anicut, on the Shimsha, is founded on rock, and is 900 feet long; it raises the water level 14 feet, and feeds eight tanks; capital outlay £9200, net returns, £4145.

The Sriramadevara anicut, on the Hemavatti, completed in 1870, has a length of 1000 feet, an average height of 22 feet, and a delivery of 400 cubic feet per second; outlay £35 000, estimated net returns £9600, at a duty of 40 acres to the cubic foot per second supply, and a water rate of 12s. per acre; this gives a percentage of 27 per cent. on the capital.

The Marchalli anicut, on the Lachmantirth, has a length of 268 feet, and raises the water 12 feet; outlay £2388, estimated returns about 27 per cent.

Catchment data of Reservoirs and Lakes in Great Britain. (Beardmore.)

Large Districts.	Description.	Height above sea.	Catchment area.	Mean Discharge of year.	Mean. Discharge per sq. mile	Representing rainfall run off.	Registered rainfall per annum.	Maximum discharge per square mile.
		Feet.	Sq. miles.	c. f. p. s.	c. f. p. s.	Inches.	Inches.	c. f. p. s.
Bann and Lough Neagh, at Weir..	Hilly	46 to 1765	22 205	3319	1·58	21·44	27·44	5·00
Brosna, at Ferbane Bridge.........	Hilly	152 to 1054	446	736	1·65	22·38	36·70	8·91
Robe, Mayo, at Ballinrobe	Flat	100 to 370	109	235	2·15	29·14	49·25	17·62
Loch Lubnaig	Precipitous ..	400 to 2500	70	417	5·97	81·33	70·6	78·63
Loch Katrine	Precipitous ..	400 to 2500	72	436	6·09	81·70	103·3	34·92
Small Hill Districts.								Storage. Millions of cubic feet.
Bann Reservoirs .... 1838 ...	Hilly	400 to 2800	5·15	18·21	3·50	48·	72·	56·0
Greenock .... 1828 ...	Flat Moor ...	512 to 1000	7·88	23·61	3·29	41·	60·	38·
Glencorse Pentland	Precipitous ..	734 to 1600	6·00	10·	1·66	22·3	37·	7·66
Belmont .... 1843 ...	Moorland ...	850 to 1600	2·81	10·5	3·74	50·7	63·4	26·8
Rivington Pike .... 1847 ...	Various	800 to 1545	16·25	48·0	2·94	40·	55·5	29·6
Turton and Entwistle ... 1836 ...	...	500 to 1300	3·18	9·61	3·02	41·	46·2	31·43
Ashton .... 1844 ...	...	800	0·59	60·78	1·09	15·5	40·0	21·0
Bute .... 1826 ...	...	200 to 350	7·80	13·65	1·75	23·9	45·4	...
Bolton	...	800 to 1600	0·80	1·67	2·09	32·7	46·0	25·6
Brockburn, Glasgow ... 1852 ...	...	400 to 800	4·30	3·49	0·79	47·4	60·4	...

* The storage effected by these reservoirs is in millions of cubic feet per square mile of catchment.

Data of Reservoirs constructed in various Countries. (Heywood.)

Country	Name of Reservoir.	Description of dam.	Surface.	Contents.	Maximum depth of water.	Maximum height of dam.	Cost.	
			Square miles.	Millions of cubic feet.	Feet.	Feet.	£	
France	Grosbois	Masonry.	...	300	...	...	144 000	Dupuit.
	Canal de Bourgogne	"	...	706	...	...	480 000	"
	Furens	"	...	56·5	164	...	64 000	Graeff.
England	Woodhead	Earth.	0·21	198	72	90	...	Bateman.
	Torside	"	0·25	236	84	100	...	"
	Rhodeswood	"	0·08	80	68	80	...	"
	Arnfield	"	0·06	33·6	52	67	...	"
	Hollingworth	"	0·02	11·7	52	70	...	"
India	Muta	Masonry.	5·25	715·	85	...	95 000	Col. Fife.
	Sholapur	"	6·33	2222	60	...	54 300	Campbell.
	Cholavaram	Earth.	...	3602	...	...	64 000	Fraser.
	Vahar	"	2·17	1732	80	84	...	Conybeare.
	Eight tanks in Rajputana	Earth and masonry.	...	624·3	15 to 27	...	8 114	Col. Dixon.
	Nagpur	Earth.	0·58	257·5	45	81	40 000	Binnie.

Spanish and French Masonry Dams. (From various sources.)

	Height.	Section.	Maximum pressure (P).	Theoretical section* for P=6.	Material.	Description of faces.	
	Metres.	S. metres.	Kilogrammes per s. centimetre.	S. metres.		Inner.	Outer.
Puentès	50	1519	7·9	1029	Rubble, faced with ashlar	Perpendicular	Stepped.
Alicante	41	1100	11·3	566	Rubble, faced with ashlar	Sloped ...	Stepped.
Val de Infierno ...	35 7	1084	6·5	391	Rubble, faced with ashlar	Perpendicular	Stepped.
Nijar	27·5	499	7·5	308	Rubble, faced with ashlar	Perpendicular	Stepped.
Elche	23·2	213	12·7	187	Rubble, faced with ashlar	Sloped ...	Sloped.
Almansa	20·7	139	14·0	141	Rubble, faced with ashlar	Perpendicular	Stepped.
Furens	50	1029	6·0	1029	Rubble ...	Curved ...	Curved.
St. Chamond ...	42	...	7·9	...	Rubble ...	Curved ...	Curved.
Bosmeleac	16·8	...	8·4	...	Rubble, faced with ashlar	Perpendicular	Sloped.
Glomel	13·9	...	9·2	...	Rubble, faced with ashlar	...	...
Grosbois	27·1	...	14·3	...	Rubble, faced with ashlar	Stepped ...	Sloped.

* *N.B.*—The theoretical section is that of the Delocre type.

Spanish Dams: Details of Scouring Galleries and Inlet Wells. (Heywood.)

Name of Dam.	Size of scouring gallery.						Method of closing the entrance.	Dimensions and shape of inlet well.		Inlet openings.				Distance from edge of dam.
	Up stream side.		Middle.		Down stream side.			Up stream side.	Down stream side.	Vertical distance apart.	Horizontal distance apart.	Size.	No. in a tier.	
	Height.	Breadth.	Height.	Breadth	Height.	Breadth								
Alicante...	8·56	5·9	9·84	10·82	19·18	13·12	Wooden door	Circular 2·6	24′ Circular	1·34	0·98	·36 × ·72	2	1·968
Elche......	6·56	5·9	...	...	10·83	7·58	Do.	Do. 3·0	8′ Do.	...	...	·54 × ·75	1	1·5
Almansa...	4·92	4·26	...	...	4·92	4·26	Do.	...	...	...	...	...	...	...
Puentès...	...	...	...	...	24·70	21·97	Do.	Curved 13·77	8·2 Rectangular	2·72	...	·92 × 1·8	3	...
Val de Infierno ...	14·76	9·62	...	...	14 76	12·30	Two wooden doors.	Curved chord of 14′ Rectangular 8·75	Rectangular 14′ × 5·3 8·20 Rectangular	9·84	...	...	2	11 8·03
Nijar	5·64	3·28	...	...	7·18	3·28	Sluice gate	Circular 8·	92′ Circular	1·47	...	...	1	11·77

N.B.—The dimensions are in English feet.

Financial Statistics of Indian Reservoirs.

Abstract of Financial Results of the Delhi and Gurgaon Irrigation Reservoirs.

Total results up to the end of each year.								During the year.			
Year.	Capital outlay.	Direct income.	Increased land revenue.	Gross returns.	Working expenses.	Net returns.	Interest charges.	Acreage irrigated.			Rainfall.
								Kharif.	Rabbi.	Total.	Inches.
1872–73	£18 338	£4684	£50 079	£54 763	£19 557	£35 206	£20 182	2108	8979	11 087	20 to 30
1871–72	18 338	4246	47 626	51 872	18 460	33 412	19 357	1857	5937	7 794	8 to 28
1870–71	18 338	3712	45 173	48 885	17 308	31 577	18 464	1306	7085	8 391	13 to 19
1869–70	18 338	3238	42 720	45 958	15 529	30 429	17 547	1580	8166	9 746	13 to 23
1868–69	18 255	3048	40 267	43 315	14 299	29 017	16 634	1021	2042	3 063	11 to 20
1867–68	17 530	2419	37 815	40 234	13 349	26 884	15 757	5347	8253	13 600	20 to 37
1866–67	17 010	2269	35 362	37 631	12 823	24 808	14 907	1620	7552	9 172	17 to 41
1865–66	17 010	2122	32 909	35 031	12 069	22 963	14 056	1943	5504	7 447	
1864–65	16 914	1776	31 342	33 118	10 337	22 781	13 211	738	2057	2 795	
1863–64	16 791	1453	29 775	31 227	8 965	22 262	12 371				
1862–63	15 251	1102	28 208	29 310	7 905	21 405	11 609				
1861–62	15 251	1006	26 641	27 646	7 348	20 298	10 846				
1860–61	15 251	721	25 074	25 795	7 137	18 657	10 083				
1859–60	14 843	694	23 507	24 201	6 916	17 285	9 341				

Abstract of Financial Results of the Bandalkand Irrigation Reservoirs.

Total results up to the end of each year.								During the year.			
Year.	Capital outlay.	Direct income.	Increased land revenue.	Gross returns.	Working expenses.	Net returns.	Interest charge.	Acreage irrigated.		Free irrigation.	Exclusive of free irrigation.
								Kharif.	Rabbi.		Total.
1872–73	£7203	£1318	£3214	£4532	£4278	£254	£1532	310	1300	Amount unknown.	1610
1871–72	7105	1099	2959	4058	3732	326	1212	306	877		1183
1870–71	7105	934	2704	3638	3261	377	893	417	529		946
1869–70	6550	746	2449	3196	2685	511	565	231	731		962
1868–69	5693	549	2199	2748	1756	992	281	347	993		1340
1867–68	3033	401	1933	2334	1409	926	129	195	598		793
1866–67	2291	288	1600	1889	986	903	15	178	715		893
1865–66	290	186	1297	1484	573	910	...	65	564		629
1864–65	...	...	1002	1002	136	866					
To end of 1864	...	...	...	...	15						

N.B.—These works having been under the charge of the collectors until lately, the correct financial condition—not even the true extent of irrigation—cannot be arrived at. The above may be considered as a very rough approximation to, or indication of, the real state of the works.

Abstract of Financial Results of Irrigation Reservoirs in Mhairwara and Ajmir in 1866.

By Lieut. F. J. Home, R.E.

Name of Tank.	Surface of tank when full.	Mean depth of whole tank.	Contents of tank when full.	Area irrigated from tank.	Amount of storage irrigated per acre.	Gross revenue due to tank.	Gross revenue per acre irrigated.	Gross value of one million cubic feet of water in tank.	Capital expended.	1866–67. Balance of		1866–67. Net percentage on capital.	
										Income.	Charge.	Gain.	Loss.
	Sq. ft.	Feet.	Cub. ft.	Acres.	Cubic feet.	£	£	£	£	£	£	£	£
1. Lusani ...	9 525 600	8·0818	76 984 185	192·09	400 771*	141	·734*	1·831	1113	762	...	6·85	...
2. Dewatan ...	6 350 400	6·3045	40 036 224	303·00	132 133	158	·521	3·950	1330	853	...	6·41	...
3. Kabra ...	7 938 000	...	57 693 384	57·96	995 400*	112	1·933*	1·943	767	660	...	8·60	...
4. KaliKankar	7 938 000	7·268	55 932 220	339·32	175 409	144	1·424	2·572	2479	124	...	0·50	...
5. Darathu ...	17 424 000	7·3368	127 836 403	448·14	285 260	147	·327	1·376	3171	...	291	...	0·92
6. Niran ...	13 939 200	7·6216	106 239 286	913·94	116 243	560	·612	5·271	3208	3865	...	12·05	...
Total...	...	...	...	...	709 945	...	1·884	16·943	12068	6264	291	...	...
	...	...	...	...	...	...	..	...	...	5973	...	...	...
Averages...	...	...	...	...	177 261	...	·471	2·824	...	...	...	4·96	...

N.B.—Figures marked thus (*) in columns 5 and 7 to be omitted in striking averages; the area irrigable from the Lusani and Kabra tanks being very small in comparison with their cubic contents.

Data of the Maisur Tank System.

Maisur River System.	Total length of main rivers with their affluents.	Amount of drainage area unintercepted by tanks.	Amount of drainage area intercepted by tanks.	Total area of each catchment basin.	Proportion of whole area under the tank system.
	Miles.	Sq. miles	Sq. miles.	Sq. miles.	Percentage.
I. Kistna River ..	611	4 814	6 217	11 031	56
II. Palar	47	...	1 036	1 036	100
III. Penner ...	167	334	1 946	2 280	85
IV. Pennar ...	32	222	1 319	1 541	85
V. Kaveri... ...	646	5 526	5 769	11 295	51
VI. Western Coast rivers ...	103	1 881	...	1 881	
Totals for Maisur and Curg	1606	12 777	16 287	29 064	56
Deduct for Curg ...	...	1 795	...	1 795	
Total for Maisur only ..	1516	10 982	16 287	27 269	60

Data of the Maisur Tank System—continued.

Maisur Tank System.	Under wet and garden cultivation.	Expenditure on repairs other than the Astagram channels.	Average yearly outlay.
	Acres.	£	£
From 1837-38 to 1841-42 ...	1 705 150	47 018	9 404
From 1842-43 to 1846-47 ...	1 849 759	43 225	8 645
From 1847-48 to 1851-52 ...	2 087 929	58 644	11 729
From 1852-53 to 1856-57 ...	2 160 309	70 021	14 004
From 1857-58 to 1861-62 ...	2 169 040	80 762	16 152
25 years' total outlay	...	299 670	11 987
25 years on channel repairs ...	...	57 537	2 301
25 years on tanks only ...	...	242 133	9 685

Brief Accounts of Indian Reservoirs.

The Delhi and Gurgaon Irrigation Works.—These works, consisting of lakes and reservoirs, have for their object the irrigation of the country south of Delhi, and in the Gurgaon and Rohtuk districts, a great deal of which is broken by small ranges of low hills. Attention was directed to these districts by the fearful famine of 1860, and the Government of the Panjab then ordered that works should be commenced to relieve the fearful destitution and starvation then existing; the country was therefore examined, and surveys and designs made by Mr. L. D'A. Jackson, then assistant engineer in sole charge, for the construction of storage reservoirs in the Gurgaon and neighbouring districts. The larger reservoirs and artificial lakes in the Delhi districts, originally constructed by the Mughal emperors, Akbar, Firoz Shah, Aurang Sḥah, and Firoz Toghlak, have been reconstructed and renewed since British occupation.

The natural basins are:—

1. The Najafgarh Jhil, filled by the Sahiba and its affluents.
2. The combined Kotila, Chandni, Malab, and Rajira Jhils.

These collect the drainage of the surrounding country, and saturate the land submerged; the water is then drawn off by escape channels, and the beds of the jhils are cultivated. The superintendence of these works was originally under Mr. Batty.

The artificial reservoirs, twenty-four in number, are formed by embanking natural ravines, or outfalls of natural lines of drainage; they have weirs and escape channels; irrigation is thus given to the lands above the embankment, which are cultivated after submersion, and to lands below by means of the supply given through the channels. The names of these reservoirs are:—

In the Delhi District.

1. Tilpat.
2. Palam.
3. Yahia Nagar.
4. Chattarpur.
5. Khirki.
6. Naryanah.
7. Toghlakabad, No. 1.
8. Toghlakabad, No. 2.
9. Bijwasan.
10. Aurangpur.
11. Ambarheri.
12. Badli.

In the Gurgaon District.

1. Tharsa.
2. Gwalpahari.
3. Ghatta.
4. Pattri Katal.
5. Kala.
6. Raisinah.
7. Bar Gujar.
8. Dahina.
9. Nand Rampur Bas.
10. Bahari.
11. Jhand Sarai.
12. Garhi Harsaru.
13. Banarsi.

Both the jhils and the storage reservoirs are entirely dependent for their supply on the annual rainfall, and many of them being shallow, the loss from evaporation is very great: unfortunately also, several of the reservoirs constructed in and shortly after 1861 were very defective, both in level and in alignment, their execution having been entrusted to native clerks of the collectors' law courts.

Even under these extreme disadvantages, the works paid in 1872-73 as much as 10¼ per cent., although the water rate was increased only two years before. Of the total acreage irrigated in 1872-73, 10 919 acres were under crops, three quarters of which were wheat, and 168 acres in grass; 7666 acres being supplied by the reservoirs, and 3421 acres by the natural jhils. The estimated value of the crops of the year was £40 207, irrespective of the plantations, which at present consist of 14 300 trees.

The Bandalkand Irrigation Works consist of five lakes and reservoirs in the Hamirpur, and seven in the Jhansi, districts; they have unfortunately remained under the control of the tax collectors, and little is known of the correct amount of land irrigated by them; a certain amount is irrigated free of water rate, although an increased land rate is levied on it. The names of the tanks and lakes are:—

	Miles of Distributaries.	Acres Irrigated.
In Jhansi.		
Kucha Bhawar ...	3½	7
Barna Sagar ...	8½	260
Kuchni	16	164
Pachwara	11	10
Total ...	39	441
In Hamirpur.		
Bejanagar, three	7	176
Desrapur, four ...	2	254

	Miles of Distributaries.	Acres Irrigated.
In Hamirpur.		
Thannah	5	246
Tikamau	1	48
Paswara tank ...	...	9
Kirat Sagar ...	¾	11
Maddan Sagar ...	¾	101
Kallian Sagar ...	¾	32
Bijanagar tank ...	...	1
Phulbagh	2	157
Bela Tal tank ...	...	135
Total ...	19¼	1170

The former works irrigate the land of thirteen villages, the latter that of sixty-one; about three-quarters of the crops grown are cereals, including rice and one-fifth sugar-cane. Some approximate financial results of these works will be found in the tabular statistics. It is in contemplation to increase the irrigation from these works to 22 000 acres.

The Agra Irrigation Works.—These works consisted mainly of the Fattahpur Sikri Basin, and its channels the Khairagarh and Barkol, which were supplied with water by the Utangant torrent. The latter rises in Jaipur, flows through Bhartpur, and enters the Agra district about 7 miles east of Fattahpur Sikri. The revenue derived was not only from the water that passed into the channels from the overflow of the Utangan, but from the cultivation of a portion of the area of the basin itself. The irrigation from these works being very irregular, and objections having been rased against them on sanitary grounds, the works instead of being improved, were abandoned in 1865. At that time the capital outlay had amounted to £22 312, and the total direct income was £11 077, independently of increased land revenue, which probably amounted to as much more; the yearly direct income varied between £400 and £1400, the working expenses from £600 to £1200. It would appear therefore that, as also in the more recent case of the Agra canal, irrigation from which is not to be allowed within 5 miles of Agra, there are some traditions of local magistrates and tax collectors that are opposed to irrigation.

The Rajputana Irrigation Works in Mhairwara and Ajmir consist of a number of reservoirs, or tanks, having banks generally of earth, though in many cases pitched or faced with rubble, and having masonry weirs and escapes: they were made or reconstructed under the orders of Colonel Dixon, the political agent, and had the beneficial effect of settling the rather troublesome population of those districts, and increasing it from 39 658 in 1835 to 130 282 in 1845; the cost on original works being according to old accounts only £24 111, from 1835 to 1846, and resulting in an increase of annual revenue of £11 300 in addition to £9680 obtained annually till then. The following are data of these works according to old accounts:—

Tank.	Surface. Acres.	Contents. Cub. Yards.	Irrigation. Acres.
Lusani ...	278	5 614 400	273
Loharwara	161	3 900 000	...
Kabra ...	182	4 302 222	204
Kalikankar	182	3 699 996	437
Durathu...	167	4 701 666	...
Sarnagar	109	2 934 688	...
Tarwaja ...	218	387 200	364
Rupana ...	25	524 080	36
Gohana ...	94	2 684 586	250

the extreme depths varying from 15 to 28 feet.

In 1867 these works were examined by an officer of considerable irrigation experience, Captain F. J. Home, R.E., and the accounts of

their financial results, which were then considered exaggerated, entirely readjusted: it is from his report therefore that the abstract of financial results given in the tabular statistics has been compiled. In consequence of the number of tanks, nine varying so considerably from that for which the more recent returns are given, namely, six, it is impossible to institute a perfect comparison between the two sets of returns; but it is perfectly evident that the gross return of 47 per cent., shown by the older returns, may be generally correct. It appears also, according to other accounts, that the total number of tanks in Mhairwara must be considerable, as they cover a total area of 8675 acres, and irrigate 14 826 acres of land.

In the other states of Rajputana still under native rulers, there have doubtless been a large number of tanks; in fact, the strong affinity of race between the Seljukian dynasties of Maisur and Rajputana would lead one to believe that there must have been a strong similarity in condition of the two countries. Maisur is still covered with tanks, and it is hence probable that Rajputana was also as much developed in this respect as its physicial conditions and limited rainfall allowed. In Udaipur there are still one or two magnificent lakes, and in Marwar, Jaipur, and Bhartpur, there are traces and ruins of large reservoirs, in some cases nearly obliterated by drift sand: the primary cause of the decay of these states was doubtless their proximity to the seat of government of the Mughal emperors, who plundered and devastated them; and it would at first sight appear surprising that under British suzerainty they have not recovered and reconstructed their large and numerous reservoirs of irrigation. The causes are probably these: these states do not yet possess the confidence of the British capitalist; and hence, in order to carry out extensive works, they would have to borrow from native bankers at an interest of 10 or 12 per cent., while the works under good management would probably eventually only pay 18, and in a partially developed state only 9 per cent.: in the second place, in order to design and execute the works really well, they would require the services of skilled civil engineers. On this latter point, difficulties are thrown in the way by British officialism. In former times, Englishmen and Europeans were prevented from entering into the service of native princes from fear of their using their skill in assisting in military operations and rebellion against the British Government: at present, although this fear can hardly be said to exist, the tradition still remains in the minds of the British political agents, many of whom prevent the native princes from engaging the services of *independent* Englishmen, and by persevering in this childishly weak

policy, put an effective bar to the development of agriculture, and consequently to the material progress of native states.

The Tanks of the Bombay Presidency are comparatively very few, and there is very little information about them available. In the district of Nimar in the Narbada valley, is the lake of Lachma, a tank three miles in circumference;—this with 105 other tanks have been restored since the British occupation The Chuli tank on the Chuli ravine, and the Mandleshwar tank on the Chapra, both in the Narbada territory, were restored in 1846 by Captain Trench. In Gujrat a reservoir project, in connection with the Tapti, intended to irrigate 194 000 acres, is being carried out. In Kandeish a storage reservoir in the Girna valley, and the Mukti reservoir, near Dhulia, are being constructed: the latter has a catchment basin of 50 square miles, which with a rainfall of $16\frac{1}{2}$ inches, will collect 477 millions of cubic feet, of which the tank will hold about 346 millions. The Hartola tank in the same district is nearly completed. In Dharwar, the Maddak tank has recently been constructed; and some storage works in the valley of the Yerla, a tributary of the Krishna, are being made. The Ekruk tank on the Adila, a tributary of the Bhima, in the neighbourhood of Sholapur, was completed in 1869, and supplied water for irrigation in 1871. The following are the data of the original project, which was carried out by F. D. Campbell, Esq., C.E.:—

Catchment area 141 square miles, minimum annual rainfall 12 inches.

Flood discharge of Adila river 37 000 cubic feet per second.

A flood lasting five days gives 11 000 cubic feet per second.

Fall of Adila river 7 feet per mile, or 1 in 754.

Area of reservoir $6\frac{1}{3}$ square miles, maximum depth 60 feet.

Contents of reservoir 2222 millions cubic feet = $6\frac{1}{2}$ inches over catchment area.

Calculated maximum velocity over waste weir 10 feet per second.

Waste weir discharge $250 \times 5 \times 10 = 12\,500$ cubic feet per second.

Total length of dam 7200 feet, including 2730 feet masonry.

Maximum height of earthwork 72 feet, or 7 feet above flood line.

Height of masonry 3 feet above highest flood, exclusive of 3 feet of parapet above.

Evaporation of 7 feet deep during eight months = 750 millions cubic feet.

Unutilized residue in bottom of tank 20 millions cubic feet.

It has three canals of discharge.

i. The lowest, perennial, 28 miles long; its head is 20 feet above the level of the bottom of the tank, having a discharge of 44 cubic feet per second, an area irrigable from it of 25 square miles, 8 months, 912 millions cubic feet.

ii. The next for a four months' supply, 18 miles long, having a discharge of 42 cubic feet per second, an area irrigable from it of 21 square miles, 4 months, 435 millions cubic feet.

iii. The next for a four months' supply, 4 miles long, having a discharge of 21 cubic feet per second, an area irrigable from it of 10 square miles, 4 months, 217 millions cubic feet. The discharge of one 4 months' channel will be compensated by the mansun supply.

The duty of water for rice alone is fixed at 96 acres per cubic foot per second, and that for all crops together at 150.

Acreage under command, 35 840 acres.

The water rate for perennial crops is 16s., and that for one season crops 8s.

The calculated cost of the works was £100 937, including 15 per cent. for establishment; the probable gross revenue will be eventually £11 820, and the cost of maintenance £2323, at 3 per cent. on the outlay; this will yield a net revenue of £9491, or 9 per cent. on the capital expended.

The Tanks of Haidarabad are extremely numerous, the whole of the eastern portion of this state, which consists of black cotton soil, is thickly studded with them. They are all of the Madras type, similar to those of the neighbouring districts of Karnul and Ballari, and are believed to be in a very bad state of repair. There are also a few large artificial lakes, as, for instance, the Hosen Sagar near Sikandarabad, and traces of others, that at one time must have supplied a large amount of irrigation. There is unfortunately no information available as to their number or effective power, Haidarabad being an independent state extremely jealous of external interference. Latterly, however, the Nizam has engaged the services of two or three English civil engineers, and it is hence very probable that he has also commenced the repair and reconstruction of these tanks, with the view of redeveloping the irrigation of his province.

The Tanks of the Central Provinces and Berar are like those of Bombay, comparatively few and generally of small size; the Kanhan reservoir project, which involves a storage reservoir covering 41 square

miles, a main canal 142 miles long, and minor channels of 400 miles in the aggregate, is still not commenced. In Berar, a fertile cotton producing province that would gain enormously from the advantages of irrigation, the tanks are few, small, and in a neglected condition: it was at one time imagined that any large storage projects for irrigation in this province would be perfectly impracticable owing to the configuration of the country; yet in 1870, three large storage reservoirs were proposed at Donad, Balapur, and Akola, as well as several smaller ones, by a civil engineer appointed by the Government of India. Most of these projects were then set aside by the provincial head of the Public Works Department, a military man totally disbelieving in the advantages of irrigation; it is, however, now probable that under future more enlightened auspices, Berar may be changed into a well irrigated and permanently prosperous province.

The Tanks of the Madras Presidency are exceedingly numerous, and some of them are of immense size. They were made under the auspices of the Telingi rajahs. In the fourteen districts of Madras there are said to be 53 000 tanks, having probably 30 000 miles of embankments, and 300 000 separate masonry works, weirs, and escapes, yielding a revenue of £1 500 000, and having a capital sunk in them of 15 millions sterling; yet in 1853 not one new tank had been made by the English, while a very large proportion of them had been allowed to fall into disrepair.

The Viranam tank, a very ancient work, has an area of 35 square miles, and an embankment 12 miles long; it is still in full operation, and secures an annual revenue of £11 453.

The Chembrambakam tank in Chingliput resembles a large natural lake, its embankment is more than three miles long, and it has six waste weirs with a total width of 676 feet of escape; it supplies 10 000 acres of rice cultivation. This tank was enlarged in 1867, at a cost of £41 000.

The Madrantakam tank at Chinglipat yielded a gross return in 1872 of £1697, and a net return of £1607 on a capital outlay, probably spent in repairs or reconstruction, of £2248.

The Kaveri-pak tank in North Arcot is also of great antiquity; it is fed from the Paler river, and has an embankment nearly four miles long, reveted with stone along its entire length; it irrigates about 7700 acres. In 1872 its banks were much damaged by an extraordinary flood, and some repairs were therefore made. There is a large number of tanks in the deltas of the large rivers of Madras, the

irrigation from which is unfortunately mixed up with that from the deltaic canals in the official reports and returns.

In fact, the paucity of trustworthy statistics of the tanks of Madras, on which the agricultural prosperity of so large a portion of India is dependent, and on the repairs of which all capital judiciously spent seems to yield from 20 to 50 per cent., is most surprising.

The Tanks of Maisur are of native origin; they are exceedingly numerous, the whole country being amply supplied with irrigation by many series or chains of them; they are, however, owing to the configuration of the country of small size, excepting in a few cases. They are in a very deteriorated condition, and have suffered greatly from silting up and want of repair and good management. The large amount of water utilized in tanks in Maisur, is indicated in the tables of the rivers of that province. It is unfortunate that the irrigated acreage due to tanks and anicuts are inseparably mixed in official records. Maisur, although it is a plateau elevated from 2000 to 3000 feet above mean sea level, has, with the exception of the Mulnad or rainy tracts of the Western Ghats, a small amount of rainfall, thus forcing water storage as an absolute necessity on its population; it, on the other hand, has the disadvantages of a sandy, and hence leaky soil, and comparatively steep surface slopes, the longitudinal slopes varying from 10 to 20 feet per mile in the flatter portions, and 60 to 80 in the steeper portions of the country, and more rapid transverse slopes; the former enhancing the cost of storage, the latter diminishing the breadth of irrigation from the channels of distribution. Stone is abundant, and is worked into rough forms, though too hard to be dressed for ordinary work. It is a gneiss of horizontal cleavage, which splits into sheets 3 to 24 inches thick, and 25 to 35 feet long, and is excellent for slabs and pillars, too hard to be dressed for ordinary work. For pitching, natural boulders are used, which are generally very round. Clay, on the other hand, is very rare; and lime is generally to be found only at great distances, and is hence often dispensed with in anicuts and overfalls, which are made to depend for stability on the size and position of the boulders.

According to the returns of 1853, there were 26 450 tanks in Maisur, of which 4106 were large irrigating reservoirs, 13 737 small, and 8609 unirrigating, *i.e.*, in a useless condition; giving about 1 effective tank per square mile in the gross; the area of Maisur being 27 269 square miles, of which 60 per cent. is under the tank system. In the seven districts of Kolar, where there are moderate conditions of rainfall,

and no very large reservoirs, there were 3611 tanks, of which 2950 were irrigating, giving 1·07 tanks to a square mile, and an approximate average quantity of wet cultivation of 10 acres to each tank. In the comparatively rainless tract, comprising portions of six districts, on which the annual rainfall varies between 10 and 20 inches, there were 1009 tanks, giving 0·31 irrigating tanks per square mile, and 2·5 acres of wet cultivation as an average to each tank. After that time a certain amount of money was spent in repairs. In 1866, however, the Executive Engineer of the Bangalor Division had reported that fully half the tanks under his charge were breached; in Chittaldrug 285, or one-third of the recorded number, were out of order; in Tomkur, 530 out of 1124; in Shemugah, 2496 out of 4520; and in the Maisur Division, 705 out of 1409. Hence, it appears, that there were in all about 1500 larger tanks requiring repair at a rate of £300 each, and 3000 smaller at £150, and that a total outlay of £900 000 was necessary to put them in good order.

In 1872–73 as many as 249 tanks were breached. The Irrigation Department of Maisur is now dealing with the matter gradually, by bringing the tanks up to a certain standard of repair, and then handing them over to the superintendence of the tax collectors; by these means it is hoped that the tanks of Maisur will be economically brought into good condition.

Among the very large reservoirs requiring special notice, are the Naggar Sulikerrai, on the river of that name, which has a margin of about 40 miles, and an embankment 1000 feet long, 84 feet high, and 600 feet breadth of base; the Maddak tank on the Vedavatti, whose embankment is 1220 feet long, and 90 feet high, having a breadth of base of 660 feet; and the Motitalao, on a feeder of the Lokani, having an embankment 117 feet high, 225 feet long, and a breadth of base of 375 feet. These are in specially favoured situations, between two hills guarding the outlets of large valleys. The proposed Mauri Cunawai and Kumbarcattai reservoirs have similar sites.

Description of an average Maisur Tank.—Length of dam ¼ to 1½ miles; 18 feet high, 12 feet top breadth, 60 feet base. Front revetment of rough stone, with a batter of 1 to 2, its facing 1·5 to 3 feet thick backed with the same thickness of loose rubble; sluices 1 to 3 to each tank; section of vent 2½ feet × 2 feet, length 30 to 120 feet, form of section sometimes barrel-shaped, sometimes rectangular; they lead off from the lowest point in the tank. Inlet cistern 3 feet high, 6 feet square, outlet cisterns the same; plug pole and gibbed stones for

orifice; escape weirs 1 to 4 for each tank, 30 to 300 feet wide, made of the largest stones, water front 3 to 9 feet deep; dam stones 3 feet apart, 4½ feet high, which when dammed give 2 feet more water; wing walls 3 to 6 feet high, converging and afterwards diverging; tail paved either sloping for a long distance or horizontal: a lower stone wall is sometimes placed across the tail at some distance off to intercept some of the escape water, which is taken off by a channel.

Waterworks of Indian Cities.

The Bombay Waterworks, by Henry Conybeare, M.I.C.E., and — Walker, C.E.

Bombay was the first of the Indian cities to carry out for itself waterworks on a modern system, and call in the aid of English civil engineers to design and superintend their execution.

In 1854, Mr. Henry Conybeare determined that the Vehar basin, in the valley of the Goper, was adequate to the collection and storage of all the water that would be required for Bombay for some years; the works were therefore confined to the formation of one artificial lake, and their execution entrusted to Mr. Walker as Resident Engineer in 1856. The catchment area was 3948, and was capable of being extended by catchwater drains to 5500 acres; the annual rainfall 124 inches, of which it was calculated that six-tenths or 74·4 inches would be available, would in these cases supply 6600 millions, or 9000 million gallons. The storage capacity allowed was 10 800 million gallons; deducting from this the loss from evaporation, which at 6 inches per month for the eight dry months of the year, would amount to 1000 million gallons, the available supply would be 9800 millions. As the annual rainfall on the gathering grounds greatly exceeded the annual consumption of Bombay, it was evident that the water would continue to rise in the lake from the commencement to the end of the rains, or for three months, leaving only nine months' consumption to be provided for. Hence, the reserve allowed in the lake, was equal to 9800 — 3700 = 6100 million gallons, at an allowance of twenty gallons per head per day for a population of 700 000 during nine months, and was thus nearly equal to two years' supply.

When filled up to the level of the waste weir, the maximum depth of the Vehar lake is 80 feet; it covers an area of 1394 acres, and stands 180 feet above the general level of Bombay. The three dams by which the water in the lake is impounded are as respectively 84, 42, and 49 feet in extreme height, and 835, 555, and 936 feet in extreme length at the top, and they altogether contain the following quantities as totals: earthwork, 406 066 cubic yards; puddle, 55 059; broken stone under pitching, 1983 cubic yards; and pitching, 53 617 square yards. The top width of dam No. 1, which carries a road, is 24 feet, and that of the two others 20 feet; the inner slope of all three embankments is 3 to 1, the outer 2½ to 1; the embankments were specified to be formed in regular layers less than 6 inches thick, watered, punned, and consolidated. The puddle walls are 10 feet

wide at the top, and batter 1 in 8; the trenches for foundations were excavated through the surface rock and past all surface springs into the solid basalt below; the slopes and tops of the dams were covered with 12 inches of stone pitching over 12 inches of broken stone.

The waste weir is 358 feet long, and has a top width of 20 feet, faced with ashlar. The water is drawn from the reservoir through a tower, provided with four inlets, at vertical intervals of 16 feet, having a diameter of 41 inches, and provided with conical plug seats faced with gun-metal—the plugs being suspended from a balcony, and worked by cranes at the top of the tower. The inlet in use is surmounted by a wrought-iron straining cage, covered with No. 30 guage copper wire, and fixed to a conical ring, fitting into the inlet orifice in the same manner as the plugs, and equally capable of being raised or lowered at pleasure: the strainer has a surface of 54 square feet. The strainer is so affixed to the cage as to admit of its being changed in ten minutes from a boat, and a plug substituted for the cage. At the bottom of the inlet well, and exactly over the entrance to the main, is another conical seat, into which a similar straining cage, having a surface of 90 square feet of No. 40 gauge copper-wire gauze is inserted. The objects of this arrangement were to utilize the whole head of water, including that due to the depth of the lake, which would have been lost had the water been strained at the outside foot of the dam; and to avoid the use of heavy sluice-valves, in positions in which it would be difficult to get at them. Without this, the utmost head obtainable would have been insufficient for a distribution by gravitation alone. No filtration arrangements nor sludge-pipe were considered necessary.

The suppy main traversing the dam is 41″ interior diameter, and its metal 1¼ inches thick: it is laid in a level trench excavated in the rock and filled with concrete: the portion traversing the puddle trench is supported on ashlar set in cement, puddled to a depth of 6 inches, and then arched over with four rings of brick in cement; two teakwood washers being affixed transversely on the pipes to prevent any water from passing between the pipes and the puddle. At the sluice-house, situated at the outside foot of the dam, the large main, 41 inches in diameter, bifurcates into two mains, each 32 inches, which continue for a distance of nearly 14 miles to Bombay. The supply is distributed through the town by branch and street mains in the usual way: the hydrants are self-closing, and of a design that admits of their closing either with or against the water pressure, the counterweights being adjusted to the resistances at the various levels of the town: the sluice-valves, 32″ diameter, are so constructed as to be capable of being closed

or opened under the severest pressure, with a very trifling exertion of force; the smaller valves are on Underhay's system, which admits of the removal of the valve seat and valve, without disturbing the laying of any portion of the mains. The water is delivered under a pressure of from 165 to 180 feet. The actual delivery of water commenced in March, 1860. The original estimate of these works was £250 000; their cost, including interest, was £655 000. The result was a supply of excellent water to Bombay of 8000 instead of 9800 million gallons daily, bringing in an annual revenue of £38 000. At present, in 1873, when the population has increased to 800 000, the supply per head amounts to only 10 gallons daily, and an additional supply is required. Various projects, having this object in view, have been proposed by Mr. Russell Aitken, Captain Hector Tulloch, and Mr. Rienzi Walton, C.E., Municipal Engineers, and a very large amount of time has been spent in discussing them.

The Madras Waterworks.

Project for the Water Supply of Madras and Irrigation near it, by W. Fraser, C.E., Executive Engineer.

The original estimate of the works was as follows:—

i. A dam across the Cortelliar stream	£3 071
ii. A channel with head and other sluices, bridges, and other requisite works, for 8½ miles, from the dam to Cholaveram tank	12 206
iii. The enlargement of this tank by raising its embankments 18 feet	15 239
iv. A channel 2¾ miles from the Cholaveram to the Red Hill tank, with sluices, bridges, and other works	6 596
v. The enlargement of this tank by raising its embankments 15 feet	11 793
vi. A channel from Red Hill tank to the Spur tank in Madras, with sluices, bridges, and other works ...	2 803
Sundries, compensation, superintendence	13 348
	£63 693

In consequence of alteration of design and increase of rates the subsequent revised estimate amounted to £104 264.

The dam as erected was 469 feet long, and 6½ feet high at crest,

resting on a solid foundation 4 feet deep, on the top of a double row of wells 9 feet deep, which were carried down to a clay stratum; the body wall was made of laterite. The head sluices consisted of ten vents 5′ × 8′ high, having piers and abutments 3 and 5 feet thick, built on 9 feet wells and 3′ foundation connected with the dam; the sill of sluices is 6 feet below the crest of the dam; these works are made of dressed gneiss and laterite. Supply channel 8½ miles long, inclination 2′ per mile, bottom breadth 30 feet, slopes 1¼ to 1, berms 15 feet each, the ordinary excavated soil to be used for embankments in low places; intended supply 100 millions of cubic yards in 35 days.

Cholaveram lake dam as existing 1 mile long, extended and raised 18 feet on hills of laterite and gravel; escape weir 200 feet long made to discharge 3½ millions of cubic yards, or a quantity equal to the total capacity of lake up to sill in twenty-four hours, with a depth of discharge on sill not exceeding 12 inches; this quantity is assumed, because these tanks have been filled in twenty-four hours of mansun in extreme cases. Supply channel in laterite, which can be utilised, section as before, fall 3 feet per mile.

Red Hill lake embankments 3000 yards long, only slightly extended, as the ground rises rapidly, and raised 15 feet. In reconstructing the embankments, the old work is stepped and the new earth laid in thin layers, sloping inwards, the puddle wall is carved up simultaneously, outside which is a 12-inch layer of gravel and stones, and beyond that 18-inch stone pitching. Surplus weir 400 feet long, to discharge and keep the surface down to 2¾ feet above sill: two irrigation sluices, and the head sluices aid in this; these are similar to those for the Cholaveram lake.

Bridges—14 road bridges; 7 foot and cattle bridges; 12 syphon culverts for under drainage and irrigation.

Data of Supply.—The Cortelliar gives 450 millions of cubic yards in 30 to 40 days of mansun; its small summer channel is perennial. Drainage area 770 square miles; the above mansun yield of which is only 6½ inches over the surface, or about one-fifth the downfall. Two other streams also yield 20 million cubic yards per annum, which is also intercepted. The Cholaveram lake formerly held 3 365 403 cubic yards, but when raised will hold 36 427 473 cubic yards. The Red Hill lake formerly held 20 477 034 cubic yards, and now 101 981 815 cubic yards; the two together 138 409 288 cubic yards; this, after deducting the amount of water to which the Mirasidars have a right, will leave 93 397 475 cubic yards; of this amount 60 millions will be used to

irrigate 8571 acres of rice, at 7000 per acre, yielding £6000, at 14s. per acre, and 33 millions for water supply. Assuming that the population of Madras will increase from 170 000 to 500 000, and will require a supply of 20 gallons per head daily, their wants will not exceed 22 million cubic yards per annum. The distribution of the town supply from the Spur tank forms a separate municipal undertaking; the municipality of Madras agreeing to pay 1 rupee per 1000 cubic yards of water taken from it.

The original rates of work per cubic yard were—earthwork of all sorts, 2¼ to 4 annas; puddling, 6 to 8 annas; revetment, 8 annas; stone work complete, 3 rupees to 3 rupees 4 annas; thus, quarrying and squaring, 1 rupee 8 annas; cartage, 2¾ miles, 1 rupee; building, 8 annas. These rates were afterwards increased.

The capital outlay up to the end of 1871–72 was £104 772, but some further sums were spent during 1872–73; from which it would seem that the Madras waterworks are now nearly in perfect working order; the income and cost of maintenance up to 1872–73, was £222 and £2911 respectively; and during 1872–73, £1516 and £667.

The Calcutta Waterworks.

Designed by W. Clark, C.E., in 1865, carried out with alterations by — Smith, C.E. The intended daily supply, 6 million gallons.

General Design.—The water is drawn from the river Hughli at Pultah, 17 miles from Calcutta, through an iron suction pipe protected from the current by an open iron jetty, the suction boxes, 36 inches, being covered with an iron sheet perforated with one-inch holes. The first engines are situated at Pultah, close to the river; they are three in number, high pressure, double acting, expansive, condensing, of 30 H. P., nominal 5 feet stroke and 30 feet lift, and pump twice a day during low water, for five hours each time, into the settling tanks close to them. The settling tanks are six in number, each being 200 × 500 feet, are used and cleaned in regular rotation: it takes one month to clean one, the deposit of mud being very large, even as much as one cubic inch to the cubic foot, or 1 part in 728 in bulk when dried. As however this has to be removed from the settling tanks in the fluid state of soft mud about three or four times daily, the above proportion of bulk amounts to or from 4000 to 5000 cubic feet of mud daily from 6 million gallons of water. For cleansing the bottoms of the settling tanks are arranged in a series of corrugations 48′ 6″ wide; on each of the ridges a drain 4′ wide by 1′ 3″ deep is formed, into which the water

is discharged. The general fall of the floor of the tank is 2 feet in the whole length of 300 feet, or enough to give the water a velocity of 2½ feet per second. The drain is dammed temporarily to cause the water to overflow and wash the sides of the ridges before flowing off in the hollows to the sludge culvert, hand-labour being employed to stir up the mud deposited. The floor is formed by excavating the earth as nearly as possible to the form of corrugation, covering it with cement concrete 6 inches thick; and over this half an inch of asphalte and then a course of brick-on-edge. The walls are built with good mortar to 5 inches less the final thickness, leaving bonding bricks projecting at intervals of a few feet; the entire surface is then plastered in cement, and the last 5 inches of brickwork built to protect it.

The apparatus for drawing water from the surface of the settling tanks is a T shaped iron tube, internal diameter 12 inches, the lower arms of the T turning, the one solid on a trunnion, the other hollow in a stuffing box, through which the water passes through the wall into the clear water culvert: the axis of the trunnion is 2 feet above the bottom of the tank, and 6 feet below full level; the arms are each 3 feet long, the revolving tube 8 feet, having its upper end protected by a spherical wire covering and attached to a float which supports it; it is raised by chain and windlass, and thrown out of gear by raising the upper end entirely out of the water.

The water rests for 36 hours in the settling tank before being drawn off into the filter beds close by them. These are designed on the downward principle, with the object of being more easily cleaned. There are eight filter beds, each 200 × 100 feet, the materials being 10 inches of coarse shingle, 4 inches of coarse sand, and 28 inches of fine sand, or 42 inches in all; the Spencer's Magnetic Carbide, originally intended, not having been used. Each filter bed can filter 1 million gallons in twenty-four hours; the whole together do filter 4½ and could filter 6 million gallons steadily, allowing two beds to be always unavailable during cleaning It usually takes from 1½ to 4 days to allow the water to filter itself before being drawn off for use. In cleaning the filter beds, the top quarter of an inch of sand is thrown away, the next inch is removed, washed and used again. The sand washing is temporarily conducted in four small cisterns, where the sand is mixed with water and stirred up with a shovel; about 50 000 cubic feet of sand can be cleaned in five weeks by this arrangement.

The water is drawn from the filter beds through the perforated brick flooring into a collecting well, whence it passes in a 42-inch cast-iron main to a covered reservoir at Tallah, about twelve miles from

Pultah, the course being generally alongside of a high road. The available fall from Pultah to Calcutta, a distance of 17 miles, is about 15 feet. This covered reservoir, intended for storage in emergency, is 400 × 200 × 20 feet, of which 16 feet is available for storage, holding thus 8 million gallons. The bottom consists of a series of inverts 15 feet span, and two rings thick, turned on a floor of 6 inches of concrete covered with a layer of asphalte. The outer walls are 2′ 6″ thick, plastered with cement.

From this reservoir engines, three in number, and similar to those at Pultah, any two being able to carry on the work, pump during the daytime the supply required for twenty-four hours for the northern division of Calcutta into the trunk-mains, and during the night-time that required for the southern division of Calcutta into a covered reservoir at Wellington Square: for both these purposes the engines lift the water from the bottom of the reservoir to a height of 50 feet above the bottom.

Distribution.—The distribution is effected from the store reservoir at Tallah in two divisions. 1st. A 30-inch inlet-main from the works at Tallah to the canal aqueduct, thence continued up to the Circular Road, 1408 yards. 2nd. A 24-inch main from the junction of Circular Road and Cornwallis Street to Wellington Square 4864 yards long. This pipe serves during the daytime as a main to supply the northern division of the town at a low pressure of 50 feet head, and at night to fill the tank at Wellington Square; whence the supply of the southern division is carried on by engines under high pressure.

The engines at Wellington Square are three in number, and of similar principle to those at Pultah and Tallah, but are of 75 H. P. nominal; any two will do the necessary work, the power being that necessary to distribute the full daily supply in six hours from the level of the bottom of the reservoir to a height of 100 feet above the surface, or a total lift of 120 feet. The work actually done by two of these engines, in ordinary practice, is to raise 132 gallons at each stroke, at a speed of 20 revolutions per minute, or in thirteen hours with three tons of fuel, to raise 3½ million gallons under a lift pressure of 60 feet.

For the low pressure division there is also an auxiliary 18-inch main, 1345 yards long, and two 12-inch mains, both together 2980 yards long. For the high pressure division, the auxiliary and lateral trunk-mains are—one 24-inch main, 220 yards long; three 18-inch mains amounting in length to 3840 yards; and ten 12-inch amounting in length to 6965 yards; exclusive of two trunk-mains 12-inch and

9-inch of 1620 and 1465 yards long respectively. The whole length of the main pipes in yards being thus:—

	30"	24"	18"	12"	9"	Valves.
Southern Division	...	220	3840	8585	1465	14
Northern Division	1408	4864	1344	2980	...	10
Total	1408	5084	5184	11565	1465	24

These mains have also district service mains in loops or sections closable by valves as follows. In the low pressure division they are 13 in number, in the high pressure division 26, consisting of the following lengths in yards:—

	9"	6"	4"	3"	Valves.
Low pressure	1830	14 908	5 414	1912	24
High pressure	2214	50 202	17 212	8336	48

The water-pipes are generally laid along the streets on the side opposite to that of the gas-pipes; they are in 9 feet lengths, and of the weights usually adopted.

The total length in yards of the mains are as follows:—

	Trunk mains.	Loop mains.	Total.
Low pressure Division	10 596	24 064	34 660
High pressure Division	14 110	42 964	57 074
Totals	24 706	67 028	91 734

or about 52 miles.

The inclinations adopted are as follows:—From Pultah to Tallah, 1 in 5500; sludge culvert, 1 in 500; river water culvert, 1 in 1600; clear water culvert, 1 in 1000.

The loop system being adopted in all future extensions or new district mains, dead ends are altogether avoided; so that on opening the valves connecting these mains with the trunk-main, a free circulation must take place throughout; the loops cannot be connected together, but additional pipes can be inserted into any of these loops to obtain an extended distribution. The pipes allowed are fully able to distribute 12 million gallons daily, or double the amount at present required. It is intended to keep the pipes constantly full under pressure, so as to obviate any necessity for cisterns.

Besides the above supply for Calcutta, the works will give eventually a supply of 120 000 gallons daily to the cantonment of Barrackpur, involving an elevated tank 50 feet high, 4660 yards of 9-inch main

pipe, and a supply of 60 000 gallons to the cantonment of Dam-Dam, under a pressure of 50 feet through 6600 yards of 6-inch pipe.

The total cost of the water delivered in Calcutta, half at 50, and half at 100 feet pressure, is estimated at about 30 000 gallons for £1.

The delivery of the main supply commenced in 1869.

The estimated prime cost was—

Price and rent of land taken for the works ...	£11 082
Machinery and Works, engines, filters, reservoirs, pipe to Tallah	377 838
Trunk and district mains, valves and hydrants, after deducting for valve of some received ...	106 676
Total...	495 596
Engineering and contingencies 15 per cent. ...	75 000
Supply to Barrackpur and Dam-Dam	10 500
	£581 096

The annual expenses are estimated at £75 964, inclusive of £57 060 for repayment of loan, at 10 per cent. on cost of works.

The Ambajhari Reservoir, constructed by A. Binnie, M.I.C.E.

The name of the projector of this scheme, which is an enlargement of a native tank, is not mentioned in the official records: it was chosen from among other projects for the supply of Nagpur, by Mr. Binnie, in 1869, and laid before Government in the two following forms:—

Project No. 1.—*Water Supply of Nagpur.*—Population, 84 000; catchment area, 6·6 square miles, bare and basaltic, having an annual rainfall 40·73 inches, mansun rainfall 37·52 inches. Proportion run off in an average mansun ·43, minimum ·268, maximum ·6.

The evaporation is based on Conybeare's measurements at Vehar, Bombay, which give 2·5 feet in eight months of dry season, or $\frac{1}{8}$ inch daily, hence allowance is made for 3·5 feet in eight months as a maximum for Nagpur. The rate of silting determined from observation to be 2·5 feet in 80 or 90 years = ·375 inches annually. Supply allowed 7 gallons per head daily, and as this is all wanted nearly at one time, the pipes are made to deliver 15 gallons per head daily. There is no filtering arrangement, but strainers of copper-wire gauze are used, being fixed in wooden frames in the inlet tower. The siphon is 2·5′ in diameter, length 185, rise 15, fall of 2′ to overcome friction: air pipe 3″ diameter. The siphon joints

are turned and bored, flanges packed with wood, bolted and fastened with hoop iron, bolts and washers. The maximum head is 78 feet or 34 lbs. per square inch, hence the pipes are tested to 130 lbs. per square inch. The formula used for the discharge of pipes is Young's Eytelwein, $v = 50 \sqrt[2]{\left(\frac{dh}{l + 50 d}\right)}$, There are scouring valves at low points. The embankment is in layers 12 inches thick, inclining inwards 1 in 6, retentive clayey material alone used; its surfaces of hard material, covered with 12″ of rough hand pitching; its slopes are outer $1\frac{3}{4}$ to 1, inner 2 to 1; its foundation is stepped and benched. The escape weir is of basalt rubble, its sill of angle-iron $3 \times 3 \times \frac{1}{2}$ welded and bolted to blocks. The waste watercourse is 18′, broad at bottom with slopes 1 to 1. The main pipe is carried on walls of rubble, or in a bed of concrete 3 feet thick, stepped into the embankment; in the valve house it is laid in concrete. Pipes above 13 inches diameter to have wide sockets, caulked with spun yarn, and lead driven in with caulking tools; those of less than 13 inches turned and bored, fixed with Roman cement. All pipes to be tested under pressure by hammer 7 lbs. weight. Angus Smith's process applied to all pipes inside and out after fitting. Distributing pipes to bear on solid ground, in trenches 4 feet to $2\frac{1}{2}$ feet deep, filled and rammed.

The puddle wall in the centre of the dam is 5 feet wide on the top, and 10 below, and 30′ high, made in layers of 8 inches.

Project No. 2, combining Irrigation with Town Supply.—Siphon as in last project; irrigation duty of water, 200 acres to 1 cubic foot per second; acreage 1121, for eight months excluding waste land = 116 225 280 cubic feet in all, including 747 acres for twelve months; distribution effected by a large irrigation pipe with wide joints, giving 7·93 cubic feet per second to start with, and decreasing in diameter so as to give only 2·32 cubic feet per second for water supply at the city 5 miles off; the intermediate points of discharge for irrigation regulating the discharge and diameter of the pipe between them: this arrangement allows 7·93 — 2·32 = 5·61 cubic feet per second for 1121 acres of irrigation, and prevents an excessive supply from being taken in the city, as it might be in an open channel. The discharges and hence the sizes of the small irrigation outlet pipes are calculated as if they were independent up to the reservoir; sluice cocks are provided at the branch outlets. A gauging and regulating apparatus, worked by a table of discharges calculated for every ·01 foot of rise for submerged orifices and weir, controls the whole supply.

The details of the above projects were drawn up in 1869, the former

being sanctioned in April 1870, and the contemplated irrigation being deferred. The estimates amounted to £32 535; the reservoir was opened in October 1872, but the distribution was not carried out by that time. The reservoir has a top surface of 370 acres, and a storage of 257·5 million cubic feet, of which 240 millions, or 1500 million gallons, are available.

The cost of excavating the puddle trench, including pumping, was £2368, at the rate of 1s. per cubic yard; the cost of puddle, £6659, at 4s. per cubic yard; the cost of embankment, in 1 foot layers rammed and watered, was £4277, at 5½d. per cubic yard; the rates for pitching were from 5s. to 10s., and for turfing, 2s. per 100 superficial feet; the total cost of the outlet, including straining-tower, foot-bridge, well and valve house, was £2893, and that of the escape weir, £821; the rates for ashlar, basalt, rubble, and concrete being from 27s to 54s., from 10s. to 16s., and 8s. per cubic yard.

The distribution source is a public one, the water standards being placed 100 yards apart along the streets. The main pipe was 4 miles long and 1·1 feet in diameter, and the distribution pipes 10 500 yards long and 1 foot in diameter; the pipes were delivered in Bombay at £7 5s. per ton, and in Nagpur, at £11 14s. The works were completed within the estimate, and a supply of 15 gallons daily per head can be maintained in years of extreme drought.

The Akola Project for Irrigation and Water Supply, by L. D'A. Jackson, A.I.C.E., Executive Engineer for Irrigation in Berar.

The proposed works consist of—

I. A reservoir formed on the Morna river by a masonry dam and earthen embankments east and west of it.

II. An irrigation channel 5 miles to the first watershed, and 3 more to the third watershed to the east of the river, and irrigation channels 15 miles to the west of the river.

III. Filter beds, drinking and bathing basins, with a fountain at the town gate of Akola, with pipes to it 1¼ miles in length.

1. *Masonry dam* 625 feet long, extreme height 36 feet; area of section of superstructure down to 30 feet $\cdot 3H^2$, and of foundation below that $21h$; strengthened by buttresses 50 feet apart from centre to centre; the wing walls rise to 8 feet above the sill level and revet the embankments, which are 8 feet wide at top, slopes 2 to 1 and 3 to 1, and have a section 10·5 H; length of eastern wing 2751, western 9057 feet.

Reservoir, extreme length and breadth about $2\frac{1}{4}$ miles, area of water-spread 2500 acres: of which 1000 are under cultivation, and on which there are only a few small huts.

Contents available for perennial irrigation, cubic feet			411 055 831
Available for town supply	„	...	58 427 360
Waste or standing water	„	...	8 843 139
Total contents	„	...	478 326 330

Beside this, there will be available for mansun irrigation in season of extreme drought at least five times the above total from the perennial flow of the river.

2. *Channel.*—Section 45 square feet, slope 1 in 3000, discharge 100 cubic feet per second below original ground level in section. In eastern channel 8 super passages in each, having section of 60 square feet and discharging 150 cubic feet per second; 8 road crossings; 2 under passages through embankments, being 2 feet pipes enclosed in masonry culverts. In western channel 9 super passages, 12 road crossings, and 2 under passages. The small trenches of distribution to be made by the landowners, aided, if necessary, by loan.

3. *Town supply.*—Pipes 4 inches in diameter, having a fall of 1 in 500, and discharging ·25 cubic feet per second. Beds and basins excavated in rock, with walling above ground. Filter bed and bathing basin each 50 feet square and 10 feet deep. Drinking basin octagonal having the length of each side 40 feet, and having a jet in the centre, the water for which will be purified by a filter on the ascending principle passing through perforated walling and tiles, then large and small pebbles, sand, and magnetic carbide.

Data.—Catchment area 220 square miles, minimum downpour 12 inches of which 6 inches run off, give 3066 million cubic feet in a year of drought, and fill the reservoir six times. The extreme flood discharge over the weir sill, using a local coefficient of 12 for the formula $Q = 12 \times 100\,(N)^{\frac{3}{4}}$, $= 67\,200$ cubic feet per second; and assuming a flood velocity of 13 feet per second, this gives a flood section of 5170 square feet. The waterway allowed is $8 \times 125 = 5000$ square feet; the measured flood sections are in support of this.

Land under water command on the east bank 45 square miles, west 30 square miles; total 75, all fertile; the perennial supply for irrigation

during the eight dry months is 410 million cubic feet, or 19·5 cubic feet per second, which at a duty of 200 acres will irrigate 3900 acres. The mansun irrigation supply for four wet months exceeds any demand that is likely to occur; the probable maximum acreage for this will be about half the irrigable area, or 20 square miles on one bank and 15 on the other, being in all 35 square miles or 22 400 acres; the channel of supply is designed to carry sufficient to irrigate the total area of 75 square miles.

Cost of Works and extension on the west bank	£31 301
Compensation and Road diversion	1 000
Establishment and contingencies 20 per cent.	6 869
	£39 170

Probable return, when the works are fully developed:—

Perennial, *i.e.*, 8 months, 3900 acres at 14s ...	£2 730
Mansun, *i.e.*, 4 months, 22 400 acres at 4s. ...	4 480
	7 210
Collection, repairs, establishment, 8 per cent. ...	577
Result, a net return on capital £40 000 of 16½ per cent.	£6 633

Or, deducting capital spent in town supply, a result of 19 per cent. on the outlay on the capital spent in irrigation, independently of the water rate charged to the town.

The classification of water rates for various crops is that adopted on the Bari Doab Canal, but the rates themselves are doubled, as the cost of labour in Berar is double that in the Bari Doab. Hence the rates assumed for Berar are,—1st class, Sugar-cane, £1 4s.; 2nd, Rice and garden produce, 19s.; 3rd, All ordinary field crops, not elsewhere mentioned, 10s.; 4th, All millets, pulses, and grass crops, 6s.; 5th, A single watering, 3s. These may be expected to yield mean rates of 14s. and 4s. at the least, as it is most probable that sugar-cane will be extensively grown; all the sugar in Berar being now imported.

Irrigated Crops, Waterings, and Water Rates.

The Watering of Crops in Spain.

The following data of Mr. George Higgin, C.E., in 1873, indicate the amount of water required for crops in the irrigated districts, where the annual rainfall excepting of Granada, is from 11 to 22 inches only.

Actual water duty on old works.	Per sec. per hect. Litres.	Per sec. per acre. C. feet.	Per c. ft. per sec. Acres.
In Valencia, from the Jucar, rice ...	2·00	·0282	35
In Valencia, from the Turia, old ...	·86	·0121	83
In Gandia, type of old	·80	·0113	88
In Murcia and Orihuela, old	·74	·0104	96
In Granada, old...	·29	·0041	244
Esla and Henares, new	·45	·0064	157
Lowest duty in Spain generally, new	·50	·0071	140

The practice of watering usual in Valencia is, for lucerne, one watering in 8 or 10 days; for maize, beans and hemp, one in 15 days; for potatoes, one in 21 days; for cereals, one in 30 days; the average amount given at one watering in ordinary soil is 500 cubic metres per hectare (7060 cubic feet per acre), and the fullest ever given is 700 (9884).

The following data of Mr. Roberts, C.E., in 1867, are strongly in support of the above.

Average actual water duty in various provinces.	Per sec. per hect. Litres.	Water duty authorized for various canals.	Per sec. per hect. Litres.
In Valencia	·25	De las Cinco Villas ...	·26
In Rioja (low clayey) ...	·20	De Tamarite	·31
In Murcia, Alicante, Aragon, and Cataluña	1·00	Del Henares	·40
		Del Esla	·61
Cereals & grass generally	·25	Del Tajo	·95
Huertas or gardens ...	·75	Del Ebro	1·00
All other lands ...	·50	De Isabella Segunda ...	·75
In extremely dry seasons	1·00		

The practice of watering is—for cereals, &c., 4 to 6 waterings yearly; for meadows 8, and for gardens 20; each watering being in practice 2 inches deep, which = 550 cubic metres per hectare, and never exceeding 2¾ inches, or 7 centimetres, which = 700 cubic metres per hectare. The average number of waterings in a year given to land in Valencia is 12.

The Watering of Crops in France.

From data given with reference to the Marseilles canal, in a paper read by Mr. George Rennie before the Institution of Civil Engineers in 1855, it seems that in Dauphiné, only one watering per week, of a depth of 3 centimetres (1·18 inches), is given on heavy lands; but on light soil, and with the object of making up for losses by filtration, the depth allowed is 10 centimetres (3·94 inches); in lower Languedoc and Rousillon, the same practice of irrigating once a week, but with a depth of 5 centimetres on heavy, and 10 on light land; in the Camtale of Provence the same as in Rousillon for field crops, but a greater quantity for garden crops. There are, however, localities in Languedoc and Provence, where this system is practised only during one or two months, or for two or three times in the year.

The irrigation furnished by the canal of St. Julian, on the Durance, to 360 hectares (889 acres), at Cavaillon, Vaucluse, was 538 272 cubic metres a week, giving a calculated depth of watering each week of 15 centimetres over that area; and this is in support of an average depth of supply actually utilized of 10 centimetres once a week.

From data given by De Cossigny in the "Notions Elementaires sur les Irrigations, 1874," the watering season, in the south of France, is from the 1st of April to the 1st of October; on ordinary land in Provence the depth of watering usually given is 8 to 10 centimetres, and this is supplied once in ten to twelve days during the six months; this amounts to a total quantity of 15 552 cubic metres = 1 litre per second per hectare, as a continuous supply: garden crops require watering once in five days, and require a supply of 2 to 3 litres per second. The extreme limits are ¼ litre as a minimum, according to M. Pareto, and 4 litres as a maximum, according to M. Mangon. For various soils the same amount of water is given at each watering, but the waterings are more or less frequent, varying from once in five days for soil four-fifths sand, to once in fifteen days for soil one-fifth sand. Summer meadows require a depth of from 5 to 10 centimetres at each watering, or a continuous supply of from ½ to 4 litres per second per hectare, or an average of from 1 to 2 litres per second; although they can, according to M. Mangon, utilize and profit from as much as from 34 to 50 litres per second. For winter meadows the minimum supply advisable, according to M. Zeller, is a depth of 13 centimetres at each watering, or a volume of 1300 cubic metres in twenty-four hours, which is 15 litres per second per hectare; the maximum which they can utilise is 1700 litres per second per

hectare, in the case of the Prairie Habeaurupt; and an average allowance is from 30 to 50 litres per second per hectare. Rice crops are considered to require a permanent depth of from 15 to 20 centimetres on them, in some cases as much 40 centimetres, and a continuous supply of 1½ to 2 litres per second as a minimum: permanent stagnation of the water is considered very unhealthy. Most crops in the South of France, more especially fodder and root crops, require or greatly profit from irrigation. Oleaginous plants and arborescent cultivation do not require it. Vines are flooded to a standing depth of 10 centimetres, and kept thus for a month in winter; this destroys the phylloxera and renders the vines more fruitful in the following summer.

The Watering of Crops in Italy.

According to old data, the usual water duty in Piedmont and Lombardy was from 60 to 80 acres to 1 cub. f. p. sec., in some cases from 90 to 100, and rarely 110. From data collected by the author in Italy, in 1872, the duty under ordinary circumstances is considered to range between 80 and 110 on the most modern works. The occasion of the execution of the Lago-Maggiore project by Signori Villoresi and Meraviglia, led to a re-examination of the subject; and data were furnished by Signor Cantoni, Director of the School of Agriculture at Milan, and by a special committee of engineers. The principle adopted is that of the French, namely, that the amount of each watering to all land should be identical, and that the number of waterings alone should vary with the soil and the crop.

The following are means of results determined by De Regis, Cantoni, and the committee. The amount necessary for meadow land at each watering is 15 045 cubic feet, of which 9160 is utilized, and 5885 is absorbed; the number of waterings given varies from one in 7 to one in 10 days, thus giving a duty of from 40 to 57 acres per cub. f. p. sec.; sandy lands requiring ·025 cub. f. p. sec. per acre, and clayey lands ·017. The amount necessary for arable land at each watering is 18 173 cubic feet, of which 9697 is utilized, and 8476 is expended: the number of waterings given varies from one in 14 to one in 20 days, thus giving a duty of from 66 to 100 acres per cub. f. p. sec.; sandy lands requiring ·015, and clayey lands ·010 cub. f. p. sec. per acre. The average of the irrigable land under the Lago-Maggiore project, amounting to 193 690 acres, requires a supply of ·012 cub. f. p. sec. per acre throughout the year, or a duty of 90 acres; the maximum duty for clayey arable land being fixed at 110 acres.

The Irrigated Crops of the Panjab and the value of an acre of produce in 1872.

	The Western Jamna Canal, 1872.		Produce per acre.	Market value in 1872.	
			lbs.	£	s.
	Sugar-cane—Saccharum officinarum	Annual	2000	7	14
	Garden produce	„	...	8	0
	Rice—Oryza sativa		1920	3	6
Fibres.	Cotton—Gossypium herbaceum		720	3	12
	Hemp—Crotalaria juncea		200	1	5
Dyes.	Indigo—Indigofera tinctoria		20	1	3
	Safflower—Carthamus tinctorius		120	3	0
	Turmeric—Curcuma longa		...	...	
Oilseeds.	Til Sesamum—Orientale		160	0	16
	Toria		640	1	10
	Mustard—Saru Sinapis campestris		400	1	0
	Linseed—Linum usitatissimum		120	0	12
	Waternuts—Teopa bispinosa		6400	8	0
Drugs and Spices.	Tobacco—Nicotiana tabacum		2800	0	18
	Poppy—Papaver somniferum		240	3	0
	Dhania—Coriander		5600	2	6
	Halaun		5600	2	6
	Ajwen—Ptychotis		400	2	0
	Methi—Trigonella fœnugrœcum		400	2	0
Cereals.	Jowar—Holcus orghum		1680	2	14
	Kangni—Italian millet		1600	2	12
	Bajra—Penicellaria spicata		1520	2	10
	Chena—Panicum miliaceum		1520	2	8
	Maize—Zea mays		1600	2	10
	Wheat—Triticum vulgare		1520	3	4
	Barley—Hordeum cœteste		1120	1	10
	Oats—Avena sativa		1200	2	8
Pulses.	Gram—Cicer arietinum		1440	2	5
	Masur—Ervum lens		400	0	12
	Urad—Delichos pilosus		1440	2	16
	Mung—Phaseolus mungo		1440	1	16
	Moth—Phaseolus aconitifolius		1440	1	16
Fodder.	Lucerne—Sinji Medicago sativa		3200	2	0
	Country grass		4800	0	15
	Charri—Holcus sorghum		3200	0	8

Seasons of Crops on the Indus Inundation Canal, Derajat, 1872.

	Usual time of		Earliest date of Watering.	Latest date of Watering.	Remarks.
	Sowing.	Reaping.			
Cotton	1st to 15th June	Oct. and Nov.	1st to 15th May	Up to date of reaping in Oct. and Nov.	Cannot be sown earlier on account of hot winds.
Rice	May	15th to 30th Aug.	1st to 15th May	15th Aug. to 18th Sept.	Sown in March, watered from wells, and transplanted in May.
Indigo	1st to 15th May	1st year August, 2nd year 1st to 15th Sept.	1st year 8th May, 2nd year in April	End of Aug.	Indigo for seed gets a watering in September.
Jowar	June	15th Sept.	15th to 31st May	15th to 30th Aug.	These crops require at least one watering before ploughing, and two after sowing.
Bajra	July	15th to 30th Sept.	1st to 15th April	1st to 15th Sept.	
Barley ...., Gram ...., Wheat	20th Sept. to 31st Oct.	20th Mar. to 15th April.	July	September	Require 1, 2, or 3 waterings before sowing, and sometimes one after sowing, but not often.

N. B.—The inundation of the Indus commences in May.

The Watering Season of the Irrigated Crops of the Panjab.

		Time of Sowing.	Time of Reaping.	Earliest date of Watering.	Latest date of Watering.
The Western Jamna Canal.					
Kharif	Sugar-cane	Feb. to April	Nov. to Feb.	1 March	28 Feb.
	Cotton	March to June	Sept. to Dec.	1 March	31 March
	Rice	May to July	Sept. to Oct.	1 May	31 Oct.
	Jowar	June to Aug.	Sept. to Oct.	1 June	15 Sept.
	Maize	May to Aug.	October	1 June	15 Sept.
Rabbi	Wheat, Barley, Gram	Oct. to Dec.	April to May	1 October	31 March
Bari Doab Canal.					
Kharif	Cotton	25 April 25 June	15 Sept. 13 Dec.	11 April 25 April	25 Sept. 20 Oct.
	Rice	26 May 26 June	15 Sept. 4 Oct.	13 May 30 June	28 Aug. 15 Sept.
	Sugar-cane	8 Feb. 11 April	5 Oct. 8 Feb.	11 Jan. 9 Feb.	28 Aug. 15 Sept.
	Indigo	25 April 15 Sept.	25 June 15 Nov.	12 June 28 July	21 Aug. 21 Sept.
	Jowar	12 May 26 June	15 Sept. 20 Oct.	12 May 11 June	27 Aug. 21 Sept.
	Maize	do.	do.	do.	do.
Rabbi	Barley	15 Oct. 27 Dec.	11 April 5 May	15 Sept. 14 Oct.	19 March
	Gram	15 Oct. 29 Oct.	31 Mar. 16 April	27 Aug. 29 Sept.	not after sowing
	Wheat	15 Oct. 29 Oct.	31 Mar. 16 April	15 Sept. 14 Oct.	24 March

Experiments in Watering Crops of Wheat and Rice on the Bari Doab Canal, by E. C. Palmer, C.E., in 1871.

The average of the experiments made and tabulated show that a average depth of 0·24 feet on the whole surface, represents a thoroug watering of the average soil of the district under consideration, an for sandy soils 0·31 feet, and the amount of water necessary for a average watering of one acre, is 0·24 × 43 560 = 10 454 cubic feet.

Wheat in a dry season requires five waterings; the first, for pr paring the land for ploughing, at 10 500 cubic feet, and four for th standing crop of 8000 cubic feet, give 42 500 cubic feet in all nece sary for each acre of wheat.

Rice requires ten floodings; the amount of water necessary fo each flooding is the amount necessary to saturate the soil, th average of which, given above, is 0·24 feet, together with 0·5 feet of standing water: or in all, 0·75 feet in depth over an ac represents the quantity of a flooding, or 0·75 × 43 560 = 32 67 cubic feet; and the quantity necessary for a crop of rice is, therefor 326 700 cubic feet.

The land under consideration principally consisted of holdings o an average of 52 acres, requiring 22 acres of Kharif, and 30 of Rabl irrigation; for such a farm an irrigating outlet or pipe 0·4 feet i diameter, working under a head of 0·4 feet, was found sufficient; th discharge being 0·3323 cubic feet per second, and allowing the farm eight days to prepare his 22 acres of Kharif ploughing, and eleve days for the 30 acres of Rabbi ploughing. As the best season for th purpose lasts about six weeks, and the outlets are allowed to flow fo eight days in the month at the utmost, this arrangement allow twelve days of constant flow during that season; and thus a singl pipe, irrigating only 2·7 acres per day of twenty-four hours for ploug ing, or 5·4 acres of standing crops, is sufficient for all the purpose required in keeping up the irrigation of a holding of 52 acres.

These data are apparently in support of the amount mentioned i official returns as the average supply per acre given on the Bari Doa Canal, 44 000 cubic feet; the latter probably including also sing waterings over a certain amount of acreage.

The Canal Plantations of the Panjab.

Western Jamna Canal.

	Number in 1872.
Kikar or Babul—Acacia arabica	394 718
Shisham—Dalbergia sissu	119 611
Shahtut Mulberry—Morus alba	72 526
Tun—Cedrela tuna	33 789
Jaman—Sizygium jambolanum	17 214
Bakain—Melia azedarach	16 764
Sirus—Acacia speciosa	16 870
Gular—Ficus cunia	11 755
Jand—Acacia leucophlœa	7 205
Nim—Azadarachta Indica	7 152
Bans—Bambusa stricta	4 911
Amb—Mangifera Indica	3 774
Shahtut China—Morus tatarica	2 130
Pipal—Ficus religiosa	2 004
Miscellaneous of 80 descriptions	...
Total of all sorts	809 797

Bari Doab Canal.

	Number in 1872.
Shisham—Dalbergia sissu	451 566
Kikar—Acacia arabica	173 124
Phulai	71 710
Mulberry	54 458
Siris—Acacia speciosa	47 292
Tun—Cedrela tuna	31 853
Plum	16 735
Jand—Prosopis spicigera	11 551
Phagara—Ficus caricoides	9 760
Mudasu	6 178
Jamun—Prunus padus	4 887
Bakain—Melia sempervirens	5 966
Aliar—Dodonnœa burmaniana	4 850
Beli—Zizyphus flexuosa	4 689
Sembal—Bombax heptaphyllum	8 013
Miscellaneous trees of 83 descriptions	...
Total of all sorts	955 567

The Crops of Orissa and their Waterings.

The Late Crops, watered between June 1 and December 1 :—

		On ground from			On ground from
1.	Sarud rice ...	April to Feb.	3.	Laghu rice ...	May to Nov.
2.	Biyali rice ...	May to Oct.			

The both Season Crops, requiring perennial watering :—

		On ground from			On ground from
1.	Sugar-cane ...	April to Mar.	3.	Yams	May to Feb.
2.	Turmeric and ginger	June to Mar.	4.	Brinjal	June to Jan.
			5.	Pan and plantain	Whole year.

The Early Crops watered between December 1 and June 1 :—

		On ground from			On ground from
1.	Dalua rice ...	Feb. to May.	*6.	Tobacco... ...	Nov. to Apr.
*2.	Wheat	Nov. to Mar.	*6.	Coriander ...	Oct. to Feb.
*3.	Barley	" "	*8.	Onions and garlic	Nov. to Jan.
*4.	Gram and peas	" "			
5.	Achua cotton	Nov. to July.	9.	Achua castor oil	Nov. to Feb.

The Dry Crops not requiring irrigation are :—

Late Crops.

1. Mandia.
2. Biri pulse.
3. Black kulthi.
4. Black mug.
5. Jute and hemp.
6. Haldiya cotton.
7. Haldiya castor oil.

Early Crops.

1. White kulthi.
2. White mug.
3. Harar chaitra.
4. Mustard.
5. Linseed.

Both Season Crops.

1. Harar nali.

*2. Til.

Pulses generally.

N.B.—The crops marked * are rarely cultivated.

The usual rotation of the dry crops is, 1st year, Biyali rice (which, like Laghu rice, can be grown without irrigation), followed by pulses kulthi, mug, linseed, or mustard ; 2nd year, cotton, turmeric, ginger, or sugar ; 3rd year, fallow.

The country cotton is an annual ; of oil seeds, castor-oil is the only one that profits from irrigation ; pulses and linseed suffer from rain ; ginger and turmeric require only one or two waterings ; sugar-cane is sometimes planted as early as February and cut in November. There is a coarse species of rice grown in swampy tracts called boro dhan. The yield of Sarud rice, the staple crop, is said to be doubled by *irrigation*, and amounts to 10 cwt. per acre.

Quantity of Water required for the Irrigation of Rice in Orissa, from the Experiments of Mr. James Kimber, C.E.

The Balagurriah Plot of 54·3 acres was irrigated by means of a shoot 1 foot square, and a field channel 700 feet long therefrom. The experiments were made in the year 1872, which had a total rainfall during the irrigating season of 53 inches. From the 7th to 14th July, 1872, the water ran with ·5 foot depth in channel, and a head of 1 foot, the discharge for those seven days being 965 584 cubic feet or 1·58 cubic feet per second; gauge readings being made four times a day on each side of the field sluices. The readings reduced and entered, were averaged to give a mean daily head; from this, the amount of opening, and the number of hours open, the daily discharge was calculated. The total results were thus:—

Total amount of water given	2 885 006	cub. ft.
Area irrigated	2 368 028	sq. ft.
Amount of water represented vertically	1·213	feet.
Number of hours irrigating	674	hours.
Duty during actual irrigation of 1 cub. ft. per sec.	46	acres.
Or actual duty on the area of 1·19 cub. ft. per sec.	54·3	acres.

A similar experiment was made on the Srimuntapor Plot, but in this instance nearly double the water actually needed was used in order to obtain as much silt as possible; this then gave a duty during actual irrigation of 1 cubic foot per second to 38 acres over forty-eight days.

In the former case, however, the irrigating period was 674 hours, or twenty-eight days. Now the works generally are designed to give the same quantity of water but spread over 120 days, hence each cubic foot of water from the canal might be made to do $\frac{120}{28} = 4$ times the duty shown in the present experiment; and taken this way, the duty capable of being effected would be $4 \times 46 = 184$ acres per cubic foot per second; or, taking an average of the two sets of experiments, of which the latter seems of little value, in combination with the former, of 152 acres per cubic foot per second. But an average of this sort cannot so well be determined from an isolated plot, as it could be from utilisation of the whole of the discharge of a completed distributary. The most useful result in this case was the absolute amount of water per acre taken from the channels, which was $\frac{2885006}{54} = 53\,406$ cubic feet in the first case, and very nearly double that in the second.

The Unirrigated Crops of Barar.

	Usual date of sowing.	Shoots after	Buds after	Crop cut after	Produce per acre, excluding straw, &c.	
The Jarayat Kharif, or early dry crops.		Days.	Days.	Days.	Average.	Max.
					lbs.	lbs.
† Cotton, Gossypium herbaceum	1 July	5	120	150	100	317
† Jowari, Holcus sorghum	10 July	7	120	150	300	630
† Bajri, Holcus spicatus	1 Aug.	4	90	105	300	450
Til, Sesamum orientale	29 Aug.	7	90	105	200	660
† Rice, Oriza sativa ...	Different	5	60	105	200	600
Ambari, Hemp	10 July	3	90	120	80 bundles.	
Baru, Flax	10 July	2	60	90	100 bundles.	
† Bhadti	10 July	5	60	75	120	
Muth	10 July	5	90	105	80	
Holag	...	5	90	120	80	
* Udidh	10 July	7	90	105	240	
* Mug, Phaseolus mungo	10 July	4	105	120	300	
* Tur	...	5	90	120	180	
† Ginger, Zingiber officinale	July	12	700	1000	1100	
Red pepper, Capsicum annuum.						
The Jarayat Rabbi, or late dry crops.						
† Wheat, Triticum vulgare	22 Sept.	5	105	135	200	330
† Tobacco, Nicotiana tabacum	Sept.	8	90	150	200	480
Kardi	25 Sept.	5	90	135	120	
Lakh	9 Oct.	5	105	135	160	
Gram, Cicer arietinum	9 Oct.	5	105	135	160	
Juwas	9 Oct.	5	105	135	80	
Masur, Ervum lens	9 Oct.	5	105	135	80	
† Vutanu	9 Oct.	5	105	135	160	
Gadmol	9 Oct.	5	105	135	80	

Rough data of increase of yield to the above crops by irrigation.

Jowari, one half more.
Bajri, one quarter more.
Til, one half more.

Rice, four times more.
Wheat, Gram, } one quarter more.

* Supplementary crops, sown among others.
† Crops that may be assisted by irrigation.

The Irrigated Crops of Barar.

Baghayat or Wet Crops grown on land perpetually irrigated or kept damp by rain.	Usual date of sowing.	Shoots after	Buds after	Crop cut after	Produce per acre, excluding straw, &c.	
		Days.	Days.	Days.	Average. lbs.	Max. lbs.
Maize, Zea mays	...	5	75	105	100	...
Pepper, Capsicum perennium	1 July	7	105	370	2000	...
Bengan or Brinjal	"	7	120	370	4000	...
Bhoimug	"	5	90	120	800	...
Ganja, Cannabis sativa ...	"	8	150	150	1600	...
Onion, Allium cepa	25 Sept.	7	37	120	...	...
Garlic, Allium sativum ...	"	5	37	120	...	...
Methi, Trigonella fenugræcum	"	7	30	120	...	...
Carrots, Daucus carota ...	"	8	75	75	...	...
Kand	"	8	135	135	1200	...
Opium, Papaver somniferum.	1 Nov.	5	75	90	10	20
Sangmurla	"	5	75	90	...	...
Rajgura...	"	5	90	120	240	...
Wheat, Triticum vulgare ...	"	5	105	120	300	...
Sugar-cane, Saccharum officinarum	March	12	300	300	1600	7500
Sang of Goor	"	7	37	75	...	...
Bhend	"	7	40	80	...	...
Karli	"	7	75	90	...	...
Turai	"	8	90	120	...	...
Kawala	"	5	90	120	...	...
Chawala	"	5	37	75	...	...
Plantain	23 May	3	360	450	400 trees.	
Pan, Piper betel	...	...	...	...	...	...
Fruit trees	...	...	...	...	...	...

Well Irrigation in Barar.

1. The following crops are watered daily in the hot season, and at intervals of from one to seven days throughout the rest of the year as required; sugar-cane, pan, plantain, bengan, sag, bhaji, and green vegetable produce; when the sugar-cane is one foot high, the supply of water is reduced.

2. The following crops are watered once in three days in the hot season, and at intervals of from three to seven days throughout the rest of the year as required: ganja, opium, onions, garlic, perennial pepper, bhoimug, fenugreek, carrots, kand, chika, chakut, sangchawali, and the common produce of small vegetable gardens.

3. The following crops are watered once in three or four days at all seasons, generally: anise, saffron, turmeric, ginger, ratalu, goradu, pendia, wangi.

4. The following crops are irrigated once a week generally: sang of goor, bhend, karli, turai, kawala, chawala, sangmurla, and rajgura.

5. The remainder are: wheat, once in fifteen days; maize, three waterings to the crop; young fruit trees, once a week; older trees, four or five times a year.

The ordinary condition of the irrigation in Berar, is thus:—

The wells have an average depth of 30 feet, and are each worked by one pair of bullocks for nine hours daily, which raise a leather bag (mot) containing 300 lbs. of water. They can thus water half an acre daily well, but for a continuance cannot keep watered more than 3 acres of ordinary irrigated crops. The prime cost of a common unreveted well is £30, the bullocks £15, gear £5, in all £50: the daily expenditure is, feed of bullocks 1s., labour of two men, at 1s. each, in all 3s.; or about £50 a year.

Produce of Crops at the Experimental Farms in Barar, 1870.

Yield of clean cotton in lbs. per acre.

	Umraoti.	Sheagaon.		Umraoti.	Sheagaon.
Banni	184	86	Hinghanghat ...	180	56
Jarri	66	150	Dharwar	14	24

Manured land yielded 430 lbs. of clean cotton per acre.

The following were the yields of other crops:—Jowari, 538 lbs.; wheat, 745; gram, 312; muth, 300; linseed, 278; peas, 408 lbs.

In ploughed land, jowari yielded 660 lbs.

The Crops of the Madras Presidency and their Seasons.

	Local name.		Sown in	Cut in
Cereals.	Cholam	Sorghum vulgare	September	December.
	Kambu	Penicillaria spicata	April	June.
	Tennai	Penisetum italicum	September	January.
	Chamai	Panicum miliaceum	July	January.
	Godambai	Triticum vulgare	July	December.
	Makkai	Zea mais	July	October.
	*Nellu	Oriza sativa	July	October.
Pulses.	Thovarai	Cajanus indicus	July	April.
	Kadalsi	Cicer arietinum	July	April.
	Ulandu	Phaseolus aureus	July	February.
	Pacha payaru	Phaseolus mungo	September	December.
	Pattani	Pisum arvense	September	December.
	Tulkapair	Phaseolus aconitifolius.	December	March.
Dyes and Drugs.	Averi	Indigofera tinctoria	November	March.
	Manjel	Curcuma longa	August	February.
	Injre	Zingiber officinale	September	February.
	Emburchai	Rubia cordifolia	October	February.
	Safflower	Carthamus tinctorius	November	March.
	Kasakasa	Papaver somniferum	October	March.
	Poghrielli	Nicotiana rustica	January	April.
Fibres.	Parati	Gossypium herbaceum	May	January.
	Ganja	Cannabis sativa		
	Allivarai	Corchorus capsularis	Six months at any time.	
	Allivarai	Linum usitatissimum		
	Janupanar	Crotalaria pincea	August	March.
	Pulchi	Hibiscus cannabinus	August	March.
Oil Seeds.	Sillamunak	Ricinus communis	August	November.
	Kadagu	Sinapis, three varieties	September	February.
	Yellu	Sesamum orientale	January	April.
	Kalamilli	Coriandrum	December	March.
Miscellaneous.	Pusani	Cucurbita maxtma	July	December.
	Pudelam	Tricosanthus	July	December.
	Kotaverai	Trigonella fœnugræum	July	October.
	Pichapallam	Citrullus	February	April.
	Valleri	Cucumis sativus	April	July.
	Molam	Cucumis melo	April	July.
	Sathakupi	Anethum sirwa	December	March.

* Many various sorts of rice are grown in the Madras Presidency; one is a cold weather crop, and another is left a long time standing; but that above-mentioned is the staple crop, its period being coincident with the rainy season.

WATER RATES AND WATERINGS.

The Panjab.

On the Bari Doab Canal, from 1862–63 to 1869–70.

		s.	d.
For all crops, per acre per crop ...	2r. 6a. 8p. or	4	10

Lift irrigation, one-half the above rate.

Since 1869–70.

		s.	d.
I.	Sugar-cane, per acre per year	12	0
II.	Rice, per acre per crop ... ... Garden produce, per acre per half year	9	6
III.	Kharif crops. Cotton, hemp, indigo, turmeric sesamum, waternuts, vegetables, orchards, fruit trees ... Rabbi crops. Wheat, barley, mixed grain, linseed, sarru, taramira, mustard, opium, tobacco, tukhmbalanga, safflower, chillies, vegetables, per acre per crop	5	0
IV.	Kharif crops. All millets, maize, and crops, not elsewhere mentioned ... ... Rabbi crops. All pulses, all grasses, fallow lands, and crops not elsewhere mentioned, per acre per crop ...	3	0
V.	Single waterings, and Rabbi crops not requiring water after December, per acre per crop	1	6

For lift irrigation, one-half the above rates.

Average supply per acre, 44 000 cubic feet.

On the Western Jamna Canal, from 1862–63 to 1866–67.

	s.	d.
On all first class lands, per acre per crop	2	3½
On all second class lands, per acre per crop	1	4

For lift irrigation, two-thirds the above rates.

Since 1866–67 the rates have been identical with those of the Ganges and Eastern Jamna canals.

On the Delhi and Gurgaon Irrigation Works, from 1862 to 1870, the rates were for grass crops, per acre, 5d.; and for all other crops, per acre, 9½d.

The North-west Provinces.

Ganges and Eastern Jamna Canals, from 1862-63 to 1865-66.

		s.	d.
I.	Sugar-cane, per acre per year	8	9½
II.	Fruit, nursery and vegetable gardens, all cultivated grasses, rice, waternuts, ajawen, and similar herbs, per acre per crop	5	0
III.	Indigo, cotton, tobacco, wheat and oats (Rabbi), per acre per crop	3	4
IV.	Barley, all pulses and millets, maize, safflower, oil seeds (Kharif), per acre per crop	2	5

From 1865-66 to 1867-68.

Gardens and all lands, taking a perennial supply, were transferred from Class II. to Class I; and the rates then became for Class I., 10s. 0d.; II., 6s. 0d.; III., 4s. 6d.; IV., 3s. 4d.

Since 1867-68, the fruit, vegetable, and nursery garden produce have been transferred again into Class II., but the rates for the various classes have otherwise remained the same as before. For lift irrigation, the rates have always been two-thirds of those by flow.

The other sources of revenue are, for watering cattle, 12s. per 100, per year; sheep and goats, 4s.; supplying tanks, rent of corn mills, sale of grass, timber, fuel, and fruit, fines for trespass, &c.

Dun Canals, from 1862-63 to 1865-66.

For garden produce, sugar-cane, and first-class rice, 2s. 6d. per acre per crop; for tea, 1s. 3d.; for wheat and inferior rice, 1s. 0d.

From 1865-66 to 1867-68.

			per acre. s.	d.
I.	Tea, sugar-cane, garden, and perennial watering,	per year	10	0
II.	First-class rice, tobacco, opium, and waternut,	per crop	6	0
III.	Indigo and cotton	per crop	4	6
IV.	Inferior rice, wheat, oats, and other crops ...	per crop	2	6

From 1867-68 to 1871-72, tea and sugar-cane remained in Class I., the garden and orchard produce being transferred to Class II.; but the rates for the various classes remained unaltered.

Since 1871–72, the rate for tea has been altered to 1s. 6d. for each watering; leaving sugar-cane alone in Class I.; the rates for other produce on some of the Dun canals has been lowered.

For lift irrigation, the rates have been always two-thirds of those by flow.

Rohilkand Canals

			per acre. s.	d.
I.	Garden and orchard	per crop	4	0
II.	Sugar-cane, tobacco, opium and waternut,	per first watering	1	0
III.	All cereals, pulses, and oilseed ...	per first watering	0	6

In Classes II. and III., half rates for every subsequent watering.

For lift irrigation, the rates are half of those for flow.

The number of waterings prescribed on the Naginah Canal is:—

For fruit gardens	per year	8 waterings.
Hemp	per crop	5 „
Rice, sugar-cane, indigo, tobacco, cultivated grasses and herbs ...	per crop	4 „
Cotton, cereals, and pulses ...	per crop	3 „

NAVIGATION TOLLS IN NORTHERN INDIA.

The Western Jamna Canal transit dues are tabulated according to a most complicated code, the rates for various sorts of timber varying from 1s. 3d. to £4 per score for the whole course of the canal, with a reduction for intermediate distances; the rates by weight being about 6d. per ton for the whole course of the canal.

The Bari Doab Canal transit dues are:—

For rafts of all sorts of timber ...	1⅛d. per £10 value at starting.
For rafts of bamboos	⅜d. per thousand.
For rafts of firewood, hemp, flax, and grass	⅛d. per 4 tons, or 100 mans.
For rafts of reeds, sirkanda ...	⅛d. per thousand bundles.

The Ganges Canal transit dues, since 1872, have been:—

		s.	d.
For boats, per month		9	0
Rafts of logs, per mile	per 100 cubic feet.		1⅛
Rafts of sleepers, &c., per mile ...	„ „		½
Rafts of bamboos, per mile	„ „		¼
Rafts of firewood, per mile	per 1000 „		¼

The Eastern Jamna Canal is very little used for navigation.

Water Rates and Waterings in Southern India.

In the Bombay Presidency there is generally a combined land and irrigation assessment. The canals are divided into three sorts, and classified according to depth of soil, in cubits of 18 inches, and with respect to their special advantages and disadvantages. No advantage is considered to arise from more than two cubits in depth of soil, as it cannot imbibe and retain more effective moisture; the disadvantages taken into consideration are the presence in the soil of kankar, coarse sand, loose or stiff soil, excess of moisture, and liability to be flooded. In a moist climate the better and worse descriptions of land are considered more on a par, the latter benefiting more from moisture than the former.

The general assessment, per acre, is as follows:—

	s.	d.		s.	d.
For unirrigated or dry crops	3	6			
For ordinary irrigated or garden crops ...	8	0			
For special irrigated crops in some places ...	14	0	to	30	0

The rates allowed on the Mukti project are:—

For sugar-cane, 56s.; for rice, 20s.; for wheat, 10s. per acre.

And those allowed on the Lakh project and Bhatodi tank are:—

For perennial, or 12 months, irrigation,	per acre	18s.
For wet and cold season, or 8 months' irrigation	„	10s.
For mansun, or 4 months' irrigation ...	„	6s.

The amount of watering considered necessary per square yard of cultivation is:—

For rice crop ...	4 months ...	½ cubic yard.
For sugar-cane ...	11 months ...	1 „

A good well will keep irrigated from four to six acres of inferior garden crop.

In the Madras Presidency there is generally a combined land and irrigation assessment. The consolidated revenue, including the water rate, is two-fifths of the value of the produce, but is sometimes less, according to the market price of rice.

The general assessment, per acre, is as follows:—

	s.	d.		s.	d.
For unirrigated or dry crops	4	0			
For rice	9	6	to	16	0
Sugar, at the same ratio, would be sometimes as much as				120	0
But the general range of assessment is from... ...	4	0	to	50	0
The water rate allowed by Government on the Tumbaddra Canal of the Irrigation Company is ...	10	0	to	12	0
In Maisur, the general rate per acre is	12	0	to	15	0

The general allowance of water to rice crops in the Madras Presidency is 1 cubic foot per second of supply to 40 acres; to sugar cane, gram, plantain, and garden crops, 1 to 120 acres; ordinary field crops are rarely grown in places where irrigation is available.

When comparing the water rates in vogue in different parts of India, the average wages of a daily labourer, or coolie, should be borne in mind. The following are approximate data:—

In Northern India	3d. to 4½d.
In Barar	6d. to 9d.
In the Bombay Presidency	6d. to 9d.
In the Madras Presidency	2¼d. to 3¾d.
In Maisur	3d. to 6d.

Description and Analysis of Water.

Proportion of Silt per 100 000 *parts of water brought down by various rivers.*

(*Reduced from Heywood's table.*)

River.	Mean Proportion		Maximum.	Minimum.	Authority.
	By bulk.	By weight.	By weight.	By weight.	
Mississippi at Carrolton...	29	55	...	...	Miss. D. Survey.
„ at Columbus ...	40	76	...	...	„ „ „
„ at the mouths...	42	80	...	...	Mr. Meade.
„ „ ...	31	58	...	...	Mr. Sidell.
„ at New Orleans	33	87	...	...	Prof. Ridell.
Nile	...	158	...	...	Mr. Horner.
„ in 1874	...	...	149	7	Mr. Fowler.
Yellow River, China ...	...	333	...	...	Sir G. Staunton.
Ganges	98	196	...	...	Mr. Everest.
Hughli, at Calcutta ..	...	68	138	25	Dr. Macnamara.
Indus	...	...	476	...	Col. Tremenheere
Irrawaddi	...	33	58	17	Mr. Login.
Po	333	...	...	...	Mr. Tadini.
Rhone, at Lyons	...	6	...	...	Mr. Surell.
„ at Arles	...	50	435	14	M. Subour.
„ at Bonn	...	6	8	5	Mr. Horner.
Garonne	13	19	75	...	Mr Baumgarten.
Meuse	...	10	21	0·7	—
Seine, at Rouen	...	2	4	...	M. Marchal.
Soane	...	4	10	0·8	—
Danube	3	...	...	...	M. Marchal.
Ouse, at Ely	...	1	4	0·1	Mr. Latham.

Analysis of the Water and Silt of the Nile in 1874 by Dr. Letheby.

Constituents per 100 000 parts.		June 8.	July 10.	August 12	Sept. 20.	Oct. 12
Actual or saline ammonia ...		0·0057	0·0129	0·0043	0·0100	0·0171
Ammonia from organic matter .		0·0114	0·0100	0·0071	0·0171	0·0143
Dissolved Matters.	Lime	4·167	3·992	4.422	4·260	2·309
	Magnesia	1·623	5·113	1·030	0·617	0·483
	Soda	1·201	0·744	0·587	0·301	0·504
	Potassa	2·475	1·062	1·501	4·120	2·348
	Chlorine	1·043	0·851	0·628	0·209	0·491
	Sulphuric acid	2·808	2·838	1·837	1·996	1·908
	Phosphoric acid	trace	...	...	...	trace
	Nitric acid	trace	...	...	...	trace
	Silica alumina and oxide of iron	0·701	0·713	1·129	1·257	1·843
	Organic matter	1·500	1·057	1·186	1·929	2·414
	Carbonic acid and loss ...	4·182	3·616	4·281	4·754	3·557
Total solid matter on evaporation		20·300	16·386	16·001	19·443	15·857
Suspended matter	Organic	0·829	9·114	18·414	5·914	4·586
	Mineral	6·086	8·729	130·743	48·343	33·214
Total suspended matter ...		6·915	17·843	149·157	54·257	37·800

The average percentage of the sedimentary deposit from all the above samples was:—

Organic matter.	14·61	Potassa	1·82
Phosphoric acid	1·78	Soda	0·91
Sulphuric acid	trace.	Alumina	6·18
Chlorine	trace.	Peroxide of iron	15·15
Lime	2·06	Silica	55·09
Magnesia	1·12	Carbonic acid and loss ...	1·28
			100·

The Nile water owes its fertilizing power not only to the quantity of ammonia, nitrogeneous organic matter, the soluble silicates of potassa and soda, and traces of phosphoric and of nitric acid in the water, but also to the sedimentary matters which are charged with phosphates and alkaline silicates.

Analysis of the Water of the Ganges and its Affluents.

Date ... ... / Place	1. Apr. 1867, near Allahabad.	2. 25 May, above Danapur.	3. May, 1867, below Khanpur	4. 4 Oct. 1867, below the Son, Danapur.	5. 21 Oct. 1867, at Allahabad.	6. 14 Nov. 1867, at Khanpur	7. 1 May, 1869, at Fattahgarh	8. 11 Sept. 1869, ½ mile above Chunar.	9. Bhagaratti, 28 Oct. 1867, opposite Barhampur	10. Son. 6 Oct. 1868, at Danapur.
Total hardness...	5·8	6·	4·8	5·8	8·26	4 5	3 7	7·0	5·35	3 5
Permanent hardness	2·5	3·25	3·5	3·9	3·2	3·2	1·8	3 1	2·73	2·8
Grains of oxygen required per million grains	·02	·016?	7·3	·036 ?	·48	·4	·7	·45	1·07	·61
Ammonia	present	...	present	...	...	0	none	traces	...	traces
Phosphoric acid	abund.	present	abund.	...	...	0	none	none	traces	traces
Nitrous acid	...	...	0	...	...	0	none	none	...	traces
Grains of nitric acid in 70 000 ...	...	traces	0	...	...	0	none	none	...	under 1gr
Total solids in 70 000 grains of filtered water...	11·0	10·9	11·06	14·8	8·4	9·2	9 1	8·75	13·05	10·22
Volatile matter	3·5	1·05	2·52	2·3	·7	·51	1 75	1 4	1·26	3·01
Mineral matter	8·4	9·85	8·54	12 1	7·7	8·69	7·35	7·35	11·79	7·21
Earthy salts, silica, oxide of iron, insoluble	4·06	6·8	5·25	7·0	5 25	7·4	4·37	6·65	8·9	5·25
Lime, as carbonate	2·9	6 7	2·52	5·1	3·15	?	3·29	4·9	3·7	3·78
Silica	...	traces	traces	traces	...	...	·7	traces	3·15	1·68
Soluble salts	4·34	3·05	3 29	5·1	2·45	1·29	2·97	·7	2·8	1·96
Chloride of sodium	1·05	1·05	·8	1·26	1·05	·42	1·05	·74	·63	·42
Sulphate of soda	1·5	2·68	1·54	2·34	?	?	1·92	traces	·4	·45
Carbonate of soda	2·0	1·07	·9	1·5	·47	·76	...	·76	·8	·44

1 and 3, by Dr. Milne; 2 and 4, Dr. Jameson; 5 and 6, Dr. Compigne; 7 and 8, Dr. Whitwell; 9, Dr. Thomson; 10, Dr. May.
The Ganges is believed to supply the best river water in India.

Analysis of the Water of the Jamna.

	1.	2.	3.	4.	5.	6.	7.
Date	28 Sept. 1866,	14 Dec. 1866,	25 April, 1867,	April, 1867,	17 May, 1867,	23 Oct. 1867,	26 June, 1868,
Place	above Delhi.	2 miles above Agra.	above Agra.	opposite Allahabad.	above Delhi.	at Allahabad.	1½ mile below Mattra.
Total hardness	4·45	6·7	8·9	8·8	4·7	8·78	4·1
Permanent hardness	2·86	2·95	1·0	4·6	3·95	2·36	2·0
Grains of oxygen required per million grains ...	·05	·35	·48	·715	·055	·6	·26
Ammonia	...	...	...	traces	0	...	...
Phosphoric acid	...	...	present	present	0	...	...
Nitrous acid	...	...	...	...	0	...	...
Grains of nitric acid in 70 000	...	...	...	...	0	...	...
Total solids in 70 000 grains of filtered water ...	11·64	14·8	16·8	21·0	10·04	11·2	13·3
Volatile matter	·72	1·2	2·8	3·5	·34	·35	·8
Mineral matter	10·92	13·16	13·3	17·5	9·7	10·85	12·5
Earthy salts, silica, oxide of iron, insoluble ...	8·62	7·64	5·53	9·1	7·16	7·01	7·6
Lime, as carbonate	3·73	6·4	unknown	4·27	4·0	6·5	4·2
Silica	·58	traces	...	...	·63	...	traces
Soluble salts	2·29	5·96	7·77	8·0	3·54	2·94	4·9
Chloride of sodium	·84	1·44	2·1	4.4	·72	1·6	1·37
Sulphate of soda	1·15	1·6	unknown	3·6	2·8	?	2·
Carbonate of soda	...	·86	unknown	4·2	1·0	95	1·8

1 and 5, by Dr. Sheppard; 2, Dr. Jameson; 3, Dr. Cameron; 4, Dr. Milne; 6, Dr. Compigne; 7, Dr. May.
The Jamna water is invariably reported to be excellent everywhere.

Analysis of the Water of the Rivers of the Panjab.

	1	2	3.	4.	5.	6.	7	8.	9	10.	11.	12.	13.	14.	15.
Date	Indus 23 June, 1868,	Indus. 24 Dec 1868,	Indus. 28 Apr. 1869,	Cabul. May, 1868,	Cabul 24 Dec 1868,	Cabul. Jan. 1870,	Ravi. 16 Dec 1868,	Jhelam. 10 May, 1869,	Satlaj. 28 Mar. 1870,	Gaggar 21 Dec. 1867,	Gaggar. 28 Nov. 1868,	Harru. 13 Oct. 1867,	Harru. 24 Nov. 1868,	Loh. 25 Sept. 1867,	Swat. January 1870,
Place	at Attock.	at Attock.	at Dera, Ismail Khan.	at Naushera.	1 mile above Naushera.	near Fort Michni.	at Mian Mir?	1½ mile below Rawalpindi.	at Bhawalpur.	at Mubarikpur.	8 miles from Amballa.	above Campbellpur.	1½ mile above Saidan Baoli.	above Rawalpindi.	near Abazai.
Total hardness	3·3	6·5	4·8	4·2	8·8	10·72	5.95	2·22	7·16	6·3	6·9	8·5	9·63	8·7	6 6
Permanent hardness ...	4·3	5·	2·5	2·8	5·2	4 75	2·92	1·27	3 55	2·5	3·7	6 7	6·7	4·7	2 2
Grs. of oxygen required per million grains	·61	·16	·69	·51	·1	·80	·29	1·88	·25	·24	·29	·5	.19	·30	·24
Ammonia	...	...	none	present	...	none	none	none	none	...	...	...	...	traces	trace
Phosphoric acid	traces	...	none	traces	traces	trace	traces	none	traces	traces	traces	...	...	present	trace
Nitrous acid	...	...	none	...	...	trace	none	none	none	...	...	...	...	present	none
Grs. of nitric acid in 70 000	...	traces	none	...	...	none	none	...	none	...	...	traces	·5	traces	none
Total solids in 70 000 grs. of filtered water	6·14	10·15	10·73	9·7	15 75	18·1	11 70	7 7	11·85	15·2	15·48	17·4	17·8	19·5	10 45
Volatile matter	·35	·42	·81	·45	·14	1·60	·64	·7	·40	·34	·7	1·2	·3	1·5	1·4
Mineral matter	4·79	9·73	9·93	9·25	15 61	16·5	10·06	7·0	11·45	14·88	14 78	16 2	17·5	18·	9·05
Earthy salts, silica, oxide of iron, insoluble	3·7	5·30	4·64	5 07	...	11·2	8·79	analysis unfinished.	6·15	10·9	9·7	...	...	14·	7 25
Lime, as carbonate	3·0	...	3 04	3·46	...	3·75	4·70		4·40	8·4	6·3	...	11·1	8·6	4 5
Silica	·3	...	unk.	...	...	present	·80		·75	...	·88	...	...	2·31	traces
Soluble salts	1·19	4 34	5·29	4 18	...	53 7	2 27		5·30	3·8	5·07	...	...	4·	1·8
Chloride of sodium	·21	·42	1·31	·6	·8	1·45	·32		1·20	·42	·75	·42	·49	·63	·45
Sulphate of soda	1·3	2·8	1·35	2 64	3·6	4·1	1·47		3·30	2·6	2·23	...	...	·8	1·4
Carbonate of soda	·72	1·4	·24	1·56	...	...	...		·95	1·3	2·5	·6	...	·9	·27

1, 2, 4, 5, 10, 11, 12, 13, and 14, by Dr. Center; 7, 10, and 11, Dr. Sheppard; 8, Dr. Thomson; 6, 8, and 15, Dr. Harvey; 9, Dr. Hutcheson. 1, water rising rapidly, nearly at its highest; 2, river at its lowest; 5, river at its lowest.

Analysis of the Water of the Rivers of Oudh, Rohilkand, and the Panjab.

	1. Gumti.	2. Gumti.	3. Gogra	4. Gogra.	5. Sai.	6. Sai.	7. Surain.	8. Surain.	9. Kur-naut.	10. Garrah.	11. Ram-ganga.	12. Ram-ganga.	13. Ganguir.	14. Tovey.	15. Kurram.	16. Bain-ganga.
Date	22 April, 1867,	26 Dec. 1868,	16 June, 1867,	11 June, 1868,	27 April, 1868,	26 April, 1868,	8 April, 1868,	16 Jan. 1869,	27 May, 1869,	3 June, 1869,	6 July, 1869,	27 July, 1869,	29 July, 1869,	2 Oct. 1870,	14 Nov. 1870,	11 May, 1870,
Place	above Lakhnau.	at Lakhnau.	1½ mile above Fyzabad	1½ mile above Fyzabad	2 miles below Rai Bareli.	opposite Rai Bareli.	at Sitapur.	at Sitapur.	1 mile above Shahjahanpur.	3 miles above Shahjahanpur.	at Bareli.	½ mile above Moradabad.	4 miles above Moradabad.	2 miles above Kohat.	5 miles from Bannu.	1½ mile above Kangra.
Total hardness	4·8	4·7	4·5	4·2	8·	8·02	7·2	6·7	8·53	10·28	3·9	3·25	4·4	15·9	8·81	3·1
Permanent hardness... ...	2·09	2·	2·6	2·7	3·2	3·2	4·2	3·8	2·54	5·2	2·36	3·15	2·9	8·27	7·11	2·04
Grains of oxygen required per million grains... ...	·11	·11	·08	...	·1	·2	...	·15	2·0	·85	·35	·4	·3	·19	·28	·59
Ammonia	0	0	...	...	...	...	traces	trace	...	...	none	none	none	none	none	none
Phosphoric acid	0	0	...	...	...	...	...	none	...	...	none	none	none	none	none	none
Nitrous acid	0	0	...	...	..	...	...	none	...	...	none	none	none	none	none	none
Grains of nitric acid in 70 000	0	trace	...	...	traces	freely present	present	inappr.	...	none	none	none	none	none	...	...
Total solids in 70 000 grains of filtered water	15·4	14·	11·2	10·85	16 8	15·4	17·0	16·1	18·76	17·67	14·87	18·37	14·0	29·4	16·8	5·6
Volatile matter	1·4	1·05	·84	·7	1·4	1·75	2·0	1·4	1·96	1·93	2·21	1·75	·87	1·26	1·4	·7
Mineral matter	·14	12·95	10·36	10·15	15·4	13·65	15·	14·7	16·8	15·75	12·67	16·63	13·13	28·14	15·4	4·9
Earthy salts, silica, oxide of iron, insoluble	9·52	8·57	8·2	8·75	4·9	4·9	10·8	11·2	12·42	8·75	9·27	14·7	11·03	14·14	7·	2·8
Lime, as carbonate	7·0	8·05	5.	8·05	4·2	4·2	9 9	10·85	6·88	5·69	5·07	7·0	49·55	9·8	5·46	1·61
Silica	trace	0	...	traces	·21	·18	·3	trace	3·5	2·27	8·5	5·25	4·9	·98	trace	...
Soluble salts	4·48	4·37	2·16	1·4	10·5	8·75	4.2	3·5	4·37	7·0	3·39	1·93	2·1	14	8·4	2·1
Chloride of sodium	1·4	·63	1·7	·63	6·3	5·2	·8	·75	1·26	1·05	·74	·63	·74	2·52	1·68	1·05
Sulphate of soda	trace	·58	traces	traces	1·9	1·6	2·2	1·48	2·89	11·23	7·69	3·53	3·58	4·36	2·16	traces
Carbonate of soda	2·28	2·28	·6	·48	1·3	1·9	1·4	1·24	3·52	3·43	nil.	·57	1·33	3·8	3·78	·76

1, 2, 3, 4, 5, 6, 7, 8, by Dr. Orton; 9, 10, 11, 12, 13, 14, 15, and 16, Dr. Whitwell.
4, very good; 7, indifferent water, after a heavy rainfall.

Analysis of the Waters of the Rivers of Central India.

	1.	2.	3.	4.	5.	6.	7.	8.	9.
Date	Narbadda. 17 June, 1867,	Narbadda. 27 April, 1868,	Narbadda. 28 June, 1870,	Morar. 26 Sept. 1867,	Morar. 18 July, 1868?	Morar. 13 August, 1868, ?	Morar. 7 Feb. 1870,	Umram. 3 April, 1868,	Betwa. 14 Nov. 1867,
Place	Marble rocks.	5 miles from Jabalpur.	at Jabalpur.	1 mile below town	above bend.	above bend.	3 miles above Morar-Bazar.	above Nagod.	5 miles from Jhansi.
Total hardness	6·9	8·5	3·3	5·1	5·9	5·4	7·7	13·3	3·4
Permanent hardness	3·6	2·5	2·3	·9	·6	1·2	5·0	2·4	1·7
Grains of oxygen required per million grains	·67	·4	·315	·6	·355	·21	·46	·59	·215
Ammonia	0	...	none	...	present	...	none	...	...
Phosphoric acid	present	present	none	present	present	present	none	...	trace
Nitrous acid	0	...	none	trace	present	...	none	...	...
Grains of nitric acid in 70 000	...	...	none	·5	trace	...	none	...	...
Total solids in 70 000 grains of filtered water	12·18	11·5	5·35	10·3	15·54	9·8	15·75	18·9	9·8
Volatile matter	2·8	2·6	·95	1·2	1·3	1·3	·91	1·9	1·
Mineral matter	9·38	8·9	4·40	9·1	14·24	8·5	14·84	17·	8·8
Earthy salts, silica, oxide of iron, insoluble...	...	...	3·60	4·9	...	5·	8·4	10·	4·7
Lime, as carbonate	9·8	5·6	1·10	3 2	6·0	3·3	5·74	8·8	2·8
Silica	...	1·4	1·35	traces	2·13	...	2·1	·42	...
Soluble salts	2 1	...	·80	4·2	...	3·5	6·44	7·	4·1
Chloride of sodium	·28	1·05	·30	1·2	3·01	1·5	1·47	1·7	1·15
Sulphate of soda	traces	...	·365	...	·47	1·6	5·11	1·9	...
Carbonate of soda	·25	·51	·27	1·3	2·03	...	traces	3·	1·14

1, by Mr. Griffith; 2 and 8, Dr. Thomson; 3, Dr. Hutcheson; 4, 5, 6, and 9, Dr. May; 7, Dr. Whitwell.

Analysis of the Water of various Canals.

	1. Canal from the Ganges. Apr. 1867, below Khanpur.	2. Canal from the Ganges. 11 Nov. 1867, above Khanpur.	3. Canal from the Ganges. 23 Aug. 1869, 3 miles above Allighar.	4. Ganges Canal. 1 April, 1870, below Rurkhi Aqueduct.	Canals of the Deyrah Dun. 5. 27 Dec. 1869, Main 2 miles above Dera.	6. 7 Jan. 1870, Branch.	7. 11 Jan. 1870, Reservoir Branch.	8. 2 Feb. 1870, Branch ½ mile above Gurka Lines.	9. Canal from the Ravi. 16 Dec. 1868, at Mian Mir.	10. Canal from the Barra. 19 May, 1867, above Peshawar.	11. Canal from the Kurram. 17 Nov. 1870, near Fort of Bannu.
Total hardness	4·35	4·7	3·2	2·14	18·1	18·63	18·42	18·32	5·2	7·68	7·9
Permanent hardness	2·86	2·8	3 2	1·31	11·94	11·00	11·77	11·27	3·7	3·45	6·9
Grains of oxygen required per million grains	·3	·65	·45	·225	·14	·145	·295	·235	·25	·825	·535
Ammonia	present	0	none	...	none	none	none	none	none	present	present
Phosphoric acid	large	0	none	none	none	none	none	none	traces	traces	none
Nitrous acid	traces	0	none	none	none	none	none	none	none	present	present
Grains of nitric acid in 70,000	0	0	none	none	traces	traces	traces	traces	none	...	...
Total solids in 70,000 grains of filtered water	6·8	8·26	8·93	5·6	52·7	...	53·1	59·92	9·40	12·841	22·4
Volatile matters	·72	·7	·87	·7	·4	...	·6	·42	·48	2·677	1·4
Mineral matters	7·35	7·56	8·05	4·9	52·3	...	52·5	59·5	8·92	10·164	·21
Earthy salts, silica, oxide of iron, insoluble	5·67	5·07	5·78	...	22·4	...	...	...	5·97	6·16	10·5
Lime, as carbonate	3·71	2·6	3·71	2·1	18·7	17·50	18·4	22·68	6·24	unknown	6·58
Silica	traces	...	2·45	traces	3·3	present	·9	traces	·4	·6	1·36
Soluble salts	1·6	2·5	2·27	...	29·9	...	...	...	2·95	4·0	10·5
Chloride of sodium	1·2	1·5	·53	1·12	·8	1·30	2·8	2·5	·56	·58	1·68
Sulphate of soda	1·28	?	2·98	none	21·67	28·4	24·95	28·78	2·52	1·3	5·04
Carbonate of soda	·33	?	·66	none	1·36	...	...	8·29	...	.126	8·08

1, by Dr. Milne; 2, Dr. Compigne; 3 and 11, Dr. Whitwell; 4, 5, 6, 7, 8, Dr. May; 9, Dr. Sheppard; 10, Mr. Center; 4, contains no iron.

Result of Analysis of the average Well Waters of Stations in Northern India, according to various Analysts.

Station.	Date of Examination.	Grains per gallon, or parts in 70 000.			Oxygen required per million parts.	Character and Remarks.
		Total Solids.	Volatile matter.	Chlorides.		
Peshawar ...	May 1868	27·4	2·7	2·0	0·50	Indifferent.
Naushera ..	May 1868	18·2	0·98	0·8	0·27	Very wholesome.
Attock	May 1868	123·3	3·8	28·0	...	Very bad.
Rawalpindi ...	Sept. 1867	28·9	3·5	0·6	0·51	Pure and good.
Mian Mir ...	Dec. 1868	59·3	1·4	3·3	0·63	Very bad.
Amritsar ...	Dec. 1869	56·2	6·1	15·6	—	Good.
D. Ismail Khan	Apr. 1868	37·2	1·5	5·8	0·47	Fair.
D. Ghazi Khan	Mar. 1869	42·7	1·8	8·7	0·62	Fair.
Delhi	...	75·0	7·8	unk.	...	Very bad.
Mattra	Dec. 1867	39·6	2·0	10·8	0·51	Fair.
Agra	Jan. 1868	45·4	4·1	11·2	0·47	Bad.
Fyzabad ...	Jan. 1867	18·6	1·3	1·8	0·17	Good.
Fattahgarh ...	Apr. 1869	34·3	2·2	4·0	0·54	Doubtful.
Alligarh ...	Aug. 1869	35·1	2·6	5·7	0·44	Very foul.
Allahabad ...	Mar. 1860	38·1	1·1	3·9	...	Fair, but hard.
Banaras ...	Dec. 1868	25·9	1·3	2·8	...	Good.
Chunar ...	Sept. 1869	34·8	1·4	4·3	...	Hard and bad.
Danapur ...	Sept. 1868	59·2	5·5	10·3	0·31	Very bad.
Barhampur ...	Nov. 1867	31·1	2·3	8·7	...	Bad.
Jabalpur ...	May 1868	21·0	1·9	4·1	0·76	Wholesome.
Jhansi	Nov. 1867	25·1	4·9	2·4	0·58	Wholesome.
Morar	Aug. 1868	29·6	2·1	5·7	0·51	Bad.

Results of Analysis of the average Well Waters of Stations in the Madras Presidency and British Burmah, by Drs. Harvey, Hastings, Sinclair, and Nicholson.

Station.	Number of Wells examined.	Date of examination.	Total Solids per 100 000.	Nitric acid. per 100 000.	Hardness.	Character.
Kattak	13	Oct. 1872	50 to 100	unknown	15° to 20°	Very salt.
Kampti	6	Oct. to Nov. 1872	40 to 70	2 to 6	15° to 20°	Fair.
Sitabaldi	2	Nov. and Dec. 1872	30 to 40	1 to 4	7° to 27°	Fair.
Sikanderabad...	27	Jan. to July, 1872	31 to 90	1 to 4	12° to 30°	Bad.
Ballari	16	Feb. to April, 1872	30 to 100	·1 to 10	15° to 40°	Bad.
St. Thos. Mount	12	Aug. to Oct. 1871	30 to 100	·1 to ·2	15° to 20°	Good.
Palaveram	8	Mar. to May, 1871	50	Under 1	15°	Good.
Punamalli	6	Nov. and Dec. 1871	30 to 70	Under 1	6° to 15°	Pure.
Vizagapatam	19	May to June, 1872	50 to 200	1 to 15	20° to 40°	Salt.
Vizianagram	8	July 1872	50 to 100	unknown	25° to 80°	Indifferent.
Barhampur	7	Sept. 1872	25 to 50	1 to 2	8° to 20°	Indifferent.
Bangalor	77	During 1871	20 to 200		Variable	Bad.
Cannanor	25	Feb. to April, 1872	15	·2 to ·5	2° to 4°	Very good.
Trichinopoly	32	June to Sept. 1872	15 to 100	·5 to ·5	10° to 20°	Indifferent.
Mangalor	4	Nov. 1872	10	unknown	4° to 8°	Good.
Quilon	3	Dec. 1872	22	unknown	2° to 5°	Good.
Palamcotta	3	Dec 1872	20 to 30	1	10°	Good.
Vellor	1	Dec. 1872	56	unknown	14°	Fair.
Burmah.						
Thayatmyo	30	Dec. 1871 to Feb. 1872	50 to 100	1 to 2	20°	Safe.
Tonghu	26	June to Sept. 1872	15 to 30	·3 to 1·5	2° to 5°	Bad.
Mulmein	14	Nov. and Dec. 1872	5 to 10	·2 to ·1	2° to 7°	Good.
Shwayghin	4	Nov. and Dec. 1872	3 to 4	·2 to ·5	1 to 1·5	Good.

Analysis of the Well Waters of Madras, by Dr. Wyndowe; Averages for each Police Division of the Town.

	1st Division.	2nd Division.	3rd Division.	4th Division.	5th Division.	Seven Wells.
No. of wells	5	8	9	14	10	10
Appearance	Clear	Clear	Clear	Clear to turbid	Various	Clear
Odour	None	None	None	None	None	Various
Taste	None	Slightly brackish	Agreeable	Various	Various	Do.
Reaction to test paper	Neutral	Alkaline	Faintly alkaline	Acid	Do.	Acid
Hardness, temporary	2 to 5½	2 to 13	3 to 8½	½ to 7	½ to 6	2 to 35
Do. permanent	14 to 16	7 to 25	9 to 18	5 to 20	7½ to 16	6 to 17
Solid ingredients in one Imperial gallon.						
Organic, in grains	3 to 5	3 to 9	4 to 9	2 to 6	2 to 8	4 to 8
Inorganic, Do.	60 to 79	27 to 242	24 to 48	10 to 61	10 to 48	19 to 64
Chlorides, Do.	40 to 60	7 to 180	10 to 24	4 to 38	3 to 29	4 to 26
Bases in solution by Carbonic Acid.						
Iron	Average	Average	Average	Above	Average	Traces
Lime	Do.	Do.	Do.	Large	Do.	Average
Magnesia	Do.	Do.	Do.	Average	Traces	Do.
In solution after boiling.						
Lime	Large	Large	Fair	Average	Fair	Various
Magnesia	Average	Do.	Large	Above	Average	Do.
Chlorides	Very large	Do.	Do.	Large	Small	Not determined
Sulphates	Large	Do.	Rather large	Do.	Fair	Large
Nitrates	None	None	None	None	None to abundant	None

N.B.—The well water of Madras is believed to be as bad as possible.

Peshawar.—The drinking water is obtained by open canal from the river Barra, which also fills reservoirs; the water is excellent, but sometimes muddy; the reservoirs are frequently drained, but contain frogs, also Typha angustifolia, Potamogeitons and Confervæ.

The Peshawar Marsh being specially renowned for its malarious effects, an account of the flora that thrive there will therefore be of interest. On the higher ground, which is covered with saline efflorescence, grow several species of Salsolaceæ, Franknia pulverulenta, Tamarix, Salix Babilonica. The ordinary plants that grow in and around the marsh are:—Epilobium, occasional; Lycopus, abundant in parts; Lippia nodiflora and Herpetis monneira, about ditches; Utricularia, rare; Eclipta erecta, not uncommon; Ranunculus aquatilis and Ranunculus sceleratus, common; Limnanthemum cristatum, a species of Lium; Typha angustifolia, abundant; Nelumbium, cultivated; Butomus, rare; Sagittaria sagittafolia, Alisma equisetum, two species of Juncus, rare. Of Sedges, the following are common:—Cypicus exaltatus, Cypicus mucronatus, Malacochæte pectinata, Scupus maritimus, Carix Wallichiana, Eleocharis palustris. The common grasses about and near the water are:—Agrostis alba, Polypagon monspeliensis, Andropogon Bradlii, Cynodon dactylon, an Arundo, a Saccharum. The following are the floating and submerged plants:—A Ceratophyllum (demersum?), Potamogeiton crispus, P. pusillus, Potamogeiton plantageneus, rare; Hydrilla verticillata, Marsilia quadrifolia, Chara, most abundant; Nitella, occasional; Confervæ, profuse. Two species of Riccia, a Semno, and an Argola, are abundant in some places.

The Well Water of the Stations of the Bombay Presidency.

Bombay.—Well water brackish, containing a large quantity of lime, also sea salt. Vehar reservoir water is considered very pure.

Sattara.—Wells and tanks in trap rock; the guinea worm is found in them.

Malligaum.—The wells require clearing from sediment once a year, and would otherwise become unwholesome.

Belgaum.—Well water clear, good, soft and wholesome, contains chlorides, sulphates of lime and magnesia, and a salt of iron. Free from taste and smell.

Ahmadabad.—The well water, after long use, is apt to induce disease of the spleen, which the river water does not; the former has a higher specific gravity than the latter.

Baroda.—Well water clear, soft, and of good quality; it contains no sulphates, phosphates or nitrates, nor any salts of lime; it is alkaline;—it contains principally chloride of sodium, also carbonate of soda, and a faint trace of lime, but no iron.

Nassirabad.—Most of the wells are so salt that they are unfit for use. The water from the same well varies considerably in saltness, being sometimes palatable, clear and hard; that from a wholesome well was found to contain, after evaporation to dryness, organic matter in the large proportion of 1 in 200, as well as chloride of sodium and sulphates of alumina and potass, besides other chlorides and sulphates.

Deesa.—Well water clear, agreeable, devoid of smell, almost free from organic matter, with an inconsiderable amount of saline or mineral ingredients.

Sholapor.—Wells supplied by percolation from the tanks; water very good, soft, pure, uninjurious, and colourless, when filtered has a specific gravity of 1000·4 and contains 30 grains of solid matter to a gallon: under microscopic examination was found to contain no organic matter beyond a little shiny film. The tanks contain Flosaquæ, as well as ordinary grasses and rushes, and among the infusoria the encapsuled amalæ oscillatoria, and œdogonium; in dry weather, when the floss decomposes, the malaria is most noxious.

Surat.—There is not a single well fit for drinking from within the station. All are impregnated with sulphuretted hydrogen.

Hyderabad in Sind.—The wells are supplied by inundation from the Indus. The water is said to be soft, good and wholesome, a few wells only brackish: yet the wells swarm with animal life. Like most wells in Sind, they may be exhausted by an ordinary Persian wheel in twelve hours.

Dharwar.—The well water has the reputation of being very good and wholesome, but also to give rise to guinea-worm among the natives.

Dhulia.—Well water good, soft, devoid of smell, of an agreeable taste, but of a rather blue colour.

Serur.—Well water hard, but good and wholesome; it contains a little lime.

Ratnagherri.—Well water very good, as soft as rainwater, and free from taste or smell.

END OF HYDRAULIC STATISTICS.

INDIAN

METEOROLOGICAL STATISTICS

FOR THE USE OF ENGINEERS.

I.—India.—Average Rainfall of 72 Stations, according to the Seasons, up to 1872 inclusive.

District ……	Burdwan.						Presidency.				Raj Shahye.						Cooch Bahar.			Dacca.				
Place ……	Bardwan.	Raniganj.	Bankura.	Suri.	Midnapur.	Contai.	Saugor Island.	Calcutta.	Kishnaghur.	Jessor.	Barhampur.	Dinajpur.	Malda.	Bauliah.	Rangpur.	Bogra.	Darjiling.	Rangbi.	Buxa.	Dacca.	Faridpur.	Mymensingh.	Sylhet.	Silchar.
Height above Mean Sea Level.	102	326	…	157	108	43	6	18	…	20	64	…	160	110	…	…	6912	..	1800	35	…	…	…	89
No. of Years.	13–16	—	15–16	10	7–9	5–6	5–6	32	9–13	11–15	15–17	9–11	15–17	11–13	9–11	10–12	10–13	6	4	12–13	10	8–9	13–17	12–14
January to May.	9·0	5·2	7·7	5·4	10·2	8·8	9·1	10·1	13·8	14·4	8·8	12·5	8·9	10·3	14·1	16·4	14·5	21·5	35·2	19·5	21·5	24·0	44·2	36·9
June to September.	45·4	45·3	40·0	45·3	44·9	50·4	59·9	49·7	37·6	44·8	39·8	65·6	39·4	45·2	68·6	64·7	101·9	144·6	204·0	48·2	48·6	73·1	99·6	73·2
October to December.	6·7	4·2	4·9	4·8	7·2	15·2	13·3	6·2	5·5	7·2	6·4	6·5	5·5	6·2	5·5	7·3	8·5	9·2	17·1	6·7	5·7	5·8	10·5	9·6
Year ………	59·	55·	53·	52·	62·	74·	82·	66·	57·	66·	55·	85·	54·	62·	88·	88·	125·	175·	256·	74·	76·	103·	154·	120·

I.—India.—Average Rainfall of 72 Stations, according to the Seasons, up to 1872 inclusive.—*Continued.*

District	Chittagong.				Bahar.						Monghyr.			Orissa.					Chota Nagpur.			Assam.		
Place	Chittagong.	Noakhali.	Tipperah.	Akyab.	Patna.	Gya.	Arrah.	Muzaffarpur.	Chapra.	Champarun.	Monghyr.	Bhagulpur.	Rajmahal.	Cattak.	Falsepoint.	Puri.	Balasor.	Sambhalpur.	Hazaribagh.	Ranchi.	Puruia.	Goalpara.	Gaohatti.	Tezpur.
Height above Mean Sea Level.	90	...	...	21	179	347	191	..	182	..	160	146	...	80	18	15	15	451	1994	...	...	386	...	...
No. of Years.	13-15	14-16	12-13	12-14	10-11	9-11	14 17	12-14	14-17	9-11	16-17	16-17	2-3	12-15	6	15-16	11-13	8-11	7-8	15-17	8-11	8-9	11-13	11-13
January to May.	16·8	16·3	23·6	12·9	3·3	3·5	4·3	4·5	3·5	3·6	3·6	5·3	4·0	5·6	8·2	5·9	11·3	2·8	3·2	5·4	3·8	20·5	20·7	20·0
June to September.	79·6	75·3	61·2	176·8	31·6	36·2	41·4	36·4	31·6	38·7	32·5	38·3	43·1	40·6	50·9	39·4	47·3	43·2	43·4	34·3	35·8	72·4	45·6	52·1
October to December.	9·3	10·0	8·7	19·3	2·7	3·4	3·2	3·6	2·7	3·4	3·9	5·0	3·2	8·0	15·5	10·2	8·6	4·4	3·8	3·5	5·1	5·8	3·8	4·7
Year	106·	102·	94·	209·	38·	43·	49·	44·	38·	46·	40·	49·	50·	54·	75·	56·	67·	50·	50·	43·	45·	99·	70·	77·

I.—India.—Average Rainfall of 72 Stations, according to the Seasons, up to 1872 inclusive.—*Continued.*

District … …	Assam.					North-west Provinces.								Central Provinces.							Ceylon.			
Place…..	Naugong.	Sibsagar.	Nazirah.	Shillong.	Cherrapunji.	Benares.	Gorukpur.	Jhansi.	Lukhnau.	Agra.	Ajmir.	Bareli.	Rurkhi.	Jabalpur.	Nagpur.	Sagar.	Hoshungabad.	Chanda.	Raipur.	Seoni.	Jaffna.	Trincomali.	Colombo.	Galle.
Height above Mean Sea Level.	…	…	…	4825	4460	263	255	836	364	551	…	570	880	1353	1025	1860	1030	690	977	2030	9	175	42	40
No. of Years.	11–13	13–15	11–12	5–6	6–13	9–10	6	5–8	6	9–10	7–8	5–6	9	12	24	16	12	7	7	11	3	3–4	4	4
January to May.	24·4	28·6	24·0	14·3	104·	2·3	3·5	3·3	2·9	2·1	2·4	4·4	6·8	1·9	3·2	2·0	1·3	3·1	2·9	2·9	12·9	19·5	32·6	25·9
June to September.	57·6	58·5	55·3	56·3	400·	34·8	44·0	29·1	41·7	22·7	19·3	36·8	30·7	50·1	38·4	46·5	43·9	43·1	44·8	41·4	5·0	9·7	14·5	19·4
October to December.	5·6	7·0	6·8	7·3	19·8	2·1	4·9	2·0	2·7	0·8	0·7	1·8	1·8	1·7	3·7	2·1	1·6	2·3	2·9	2·6	32·1	30·3	33·1	28·5
Year ………	87·	94·	86·	78·	524·	37·	52·	34·	47·	26·	22·	43·	39·	54·	45·	51·	47·	48·	51·	47·	50·	59·	80·	74·

II.—Bengal.—Mean Monthly Rainfall for 16 Places before 1861.

Place	Puri.	Thayat Myo.	Midnapur.	Calcutta.	Fort William.	Hoshangabad.	Hughli.	Hazaribagh.	Barhampur.	Chunar.	Bhagalpur.	Uttramullay.	Cherrapunji.	Battikot.	Enteshwar.	Darjiling.
Latitude	19° 48′	20° 18′	22° 25′	22° 34′	22° 34′	22° 44′	22° 58′	24° 0′	24° 5′	23° 5′	25° 14′	24° 55′	25° 16′	26° 40′	26° 46′	27° 3′
Longitude	85° 49′	92° 46′	87° 19′	88° 25′	88° 25′	77° 44′	88° 26′	85° 24′	88° 17′	83° 0′	87° 0′	85° 20′	91° 44′	91° 59′	79° 5′	88° 18′
Height	15	240	...	81	18	1030	...	1900	76	250	146	4600	4500	...	...	7000
No. of Years	1	1	1	15	5	1	1	3	3	10	1	3	2	2	1	3
Actual Years	1851	1859	1851	1836–50	1855–59	1849	1851	1858–60	1857–59	1850–59	1851	1844–46	1859–60	1847–48	1848	1857–59
January	·0	·0	·3	·4	·3	1·5	1·0	·0	·1	·5	·8	4·3	·0	1·5	·0	·0
February	·4	·0	·2	·6	·4	·0	·6	1.3	·5	1·5	1·5	·2	1·8	1·2	·0	2·5
March	·0	·3	1·5	·8	1·6	·0	·0	2·0	·5	·8	·0	4·9	3·	1.1	·0	1·8
April	1·0	·5	1·7	1·9	1·6	·1	2·1	·7	1·4	·5	·8	3·6	15·9	6·5	·0	4·1
May	·5	2·3	·0	4·1	6·0	·7	·0	4·1	6·1	·5	·8	19·7	9·5	1·5	2·6	8·5
June	4·5	16·5	3·1	10·6	10·9	9·1	5·8	12·1	6·9	10·	10·5	37·0	243·3	·9	6·0	20·9
July	14·3	6·9	4·4	10·6	14·1	·9	9·6	15·5	13·6	14·	14·4	36·7	143·3	1·3	19·8	30·1
August	7·4	9·8	4·1	13·5	15·2	·0	4·2	16·6	9·7	12·	3·5	23·5	103·1	1·7	5·0	31·8
September	4·4	4·8	·0	9·0	9·6	·0	2·7	12·5	5·9	6.	2·4	7·1	77·9	2·6	1·8	14·2
October	...	6·8	7·4	4 0	5·4	·0	10·0	6·5	4·9	2·5	7·9	50·2	17·5	15·2	2·3	7·7
November	...	·0	·0	1·0	·0	·0	·0	·0	·0	·5	·0	24·7	·0	15·1	·2	·4
December	...	4	·0	·3	·0	·0	·0	1·6	·2	1·3	·0	18·0	·0	8·3	·0	·5
Year	33·	48·	23·	57·	65·	11·	36·	73·	50·	50·	43·	230·	615·	57·	38·	123·

II.—Bengal.—Mean Monthly Rainfall for 48 Places, for more than 10 Years up to 1869 or up to 1873 inclusive.

	Burdwan.					Presidency.			Rajshahye.					
Place	Bardwan.	Raniganj.	Bankura.	Suri.	Midnapur.	Calcutta.	Kishnaghur.	Jessor.	Barhampur.	Dinajpur.	Malda.	Bauliah.	Rangpor.	Bograh.
Latitude ...	23° 14′	23° 40′	23° 13′	23° 52′	22° 25′	22° 33′	23° 24′	23° 9′	24° 6′	25° 37′	25° 0′	24° 23′	25° 43′	24° 50′
Longitude ...	87° 54′	87° 12′	87° 6′	87° 35′	87° 19′	88° 21′	88° 22′	89° 7′	88° 17′	88° 41′	88° 9′	88° 33′	89° 15′	89° 10′
Height	97	326	...	157	108	18	...	20	65	...	160	110	..	...
No. of Years	13	7	13	10	10	32	10	12	...	10	14	10	7	10
Actual Years	before 1869	before 1873	before 1869	before 1872	before 1873	before 1869	before 1869	before 1869	various. old.	before 1869	before 1869	before 1869	before 1869	before 1870
January	0.6	.2	.4	.4	.7	.5	6	.3	.4	.1	.9	.2	.2	.4
February	1.2	.5	1.0	.7	.3	.7	1.0	.5	.9	.6	1.0	1.4	.3	1.0
March	1.5	1.1	1.5	.7	1.8	1.2	.8	1.6	1.2	.6	1.0	1.6	.9	1.0
April	1.9	1.1	1.9	.9	1.7	2.2	4.9	3.9	2.4	3.1	1.9	2.7	3.3	5.1
May	4.4	2.3	2.7	2.7	5.9	5.4	8.3	7.3	4.2	9.7	3.6	6.2	9.9	9.5
June	11.2	9.5	10.3	8.3	12.0	11.8	11.8	12.6	9.2	18.9	8.6	12.0	23.4	16.4
July	13.3	14.0	12.5	12.4	11.5	13.3	9.1	10.9	10.2	17.5	9.6	12.9	16.5	20.8
August	11.6	11.1	9.9	12.0	11.1	14.2	9.7	10.3	9.6	14.4	9.8	8.8	14.0	12.7
September ...	8.9	8.9	8.2	8.6	9.2	10.4	7.9	9.8	8.5	14.8	10.1	10.7	11.4	15.6
October	5.7	3.3	4.2	4.5	6.1	5.3	5.2	6.5	6.1	5.8	4.4	6.3	4.5	6.0
November ..	0.6	3	.2	.1	.3	.7	.4	1.0	.2	.3	.3	.4	.4	1.7
December ..	0.6	.0	.1	.2	.0	.2	.2	.0	.1	.0	.6	.1	.2	.1
Year	61.	53.	53.	52.	61.	66.	60.	65.	53.	86.	52.	63.	85.	90.

	Cooch Bahar.			Dacca.					Chittagong.	
Place	Darjiling.	Ranghi.	Buxa.	Dacca.	Faridpur.	Mymensingh.	Silhet.	Silchar.	Chittagong.	Noakhali.
Latitude ...	27° 3′	...	26° 48′	23° 43′	23° 35′	24° 49′	24° 55′	24° 49′	22° 21′	22° 58′
Longitude ...	88° 18′	...	89° 30′	90° 27′	89° 45′	90° 24′	91° 50′	92° 50′	91° 5′	91° 10′
Height	6941	5000	1800	35	...	...	...	89	90	...
No. of Years	10	5–6	3–4	10	4	4	14	11	12	13
Actual Years	before 1869	before 1873	before 1873	before 1869	before 1873	before 1873	before 1869	before 1869	before 1869	before 1869
January	.7	.8	.4	.4	.0	.0	.2	.5	.4	.4
February	1.6	1.5	.8	.9	.6	.4	1.6	3.5	1.6	.8
March	1.6	2.7	1.5	1.3	1.8	.6	4.9	6.1	1.3	1.6
April	3.6	6.4	7.2	7.5	5.9	3.5	14.4	12.7	5.5	4.3
May	7.0	10.5	18.7	9.8	10.9	8.0	25.0	16.1	9.4	9.1
June	27.5	31.2	55.5	13.9	14.2	21.4	30.9	19.5	22.9	21.3
July	29.4	52.4	63.3	13.4	11.6	14.9	25.9	24.6	22.5	16.6
August	29.1	34.1	37.8	12.3	14.0	15.8	23.6	16.8	23.0	19.6
September ...	18.1	27.1	32.9	8.2	10.5	10.1	13.5	13.9	13.0	14.9
October	6.6	8.5	17.7	6.3	4.6	5.4	8.7	7.8	5.9	7.9
November ..	.2	.6	1.0	1.1	.1	.0	.8	1.0	2.3	1.8
December ..	.1	.2	.1	.1	.0	.1	.2	.8	.5	.0
Year	130.	176.	238.	75.	74.	80.	150.	123.	108.	98.

II.—Bengal.—Mean Monthly Rainfall for 48 Places for more than 10 years up to 1869 or up to 1873 inclusive.—*Continued.*

Place	Chittagong. Tipperah.	Akyab.	Bahar. Patna.	Gya.	Arrah.	Chapra.	Champarun.	Monghyr. Monghyr.	Bhagulpur.	Cattak. Cattak.	False Point.
Latitude	23° 27′	20° 8′	25° 37′	24° 42′	25° 32′	25° 48′	...	25° 22′	25° 14′	20° 29′	20° 20′
Longitude	91° 5′	92° 57′	85° 8′	85° 2′	84° 46′	84° 50′	...	86° 30′	87° 0′	85° 54′	86° 47′
Height	..	21	179	347	191	182	..	160	146	80	59
No. of Years	10	11	10	10	14	14	10	14	14	12	4
Actual Years	before 1869	before 1869	before 1871	before 1871	before 1869	before 1869	before 1871	before 1869	before 1869	before 1869	before 1873
January	·8	·4	·5	·9	1·0	·8	·2	·4	·5	·5	·0
February	1·1	·4	·8	·5	·6	·6	·3	·7	·8	·5	·2
March	2·3	·5	·3	·7	·7	·5	·9	·4	·5	1·2	1·3
April	9·3	1·4	·5	·6	·9	·5	·4	·4	1·0	1·4	2·7
May	11·5	8·0	1·1	·9	1·2	1·3	1·7	1·7	2·8	1·6	2·6
June	20·4	65·2	6·8	7·0	7·7	6·3	8·1	6·0	8·6	9·6	13·7
July	17·1	55·8	10·6	12·5	14·4	8·6	10·5	10·4	11·6	11·0	13·7
August	13·8	41·7	7·4	10·1	8·9	7·7	11·2	7·9	11·2	11·2	11·0
September	10·4	24·9	7·3	7·4	10·6	6·0	8·2	7·3	8·4	9·0	10·6
October	7·0	14·7	2·6	3·4	2·7	2·6	3·6	3·7	5·3	5·6	15·3
November	2·2	6·3	·1	·1	·3	·0	·0	·1	·0	·9	4·0
December	·1	·2	·1	·1	·0	·0	·1	·1	·1	·8	·3
Year	96·	219·	38·	44·	49·	35·	45·	39·	51·	51·	[illegible]

Place	Cattak. Puri.	Balasor.	Sambhalpur.	Chota Nagpor. Hazaribagh.	Ranchi.	Purulia.	Asam. Goalpara.	Gaohatti.	Tezpor.	Naogong.	Sibsagur.	Shillong.	Cherrapunji.
Latitude	19° 48′	21° 31′	21° 28′	24° 0′	...	25° 48′	26° 11′	26° 11′	26° 36′	...	...	25° 32′	25° 16′
Longitude	85° 49′	86° 58′	84° 5′	85° 24′	...	87° 33′	90° 40′	91° 47′	92° 50′	...	...	91° 50′	91° 44′
Height	15	15	451	2014	...	...	386	...	...	...	...	4825	4460
No. of Years	13	10	10	9	14	4	10	19	10	10	14	8	11
Actual Years	before 1869	before 1869	before 1871	before 1873	before 1869	before 1873	before 1873	before 1869	before 1869	before 1869	before 1869	before 1873	before 1869
January	·1	1·2	·5	·3	1·0	·2	·3	·7	·7	1·4	1·1	·1	·5
February	1·5	1·3	·6	·4	1·0	·3	·5	1·4	1·0	1·4	2·4	·6	3·6
March	·7	2·2	·6	1·3	1·7	1·2	1·7	1·5	1·9	3·0	3·8	1·5	3·9
April	1·4	2·3	·4	·4	·6	1·4	5·5	7·3	6·8	5·9	10·1	3·7	36·7
May	2·6	4·9	1·1	1·2	1·4	4·3	12·3	10·9	9·5	11·0	11·0	8·6	76·7
June	7·9	12·8	10·0	8·1	6·6	7·7	26·5	13·3	13·8	11·9	15·6	16·0	118·9
July	9·0	8·9	13·1	14·9	10·1	14·8	19·4	13·1	16·5	15·1	14·9	14·1	157·3
August	12·4	12·6	10·5	13·6	9·0	13·8	11·9	11·9	13·1	13·2	13·9	10·1	80·0
September	9·6	13·8	7·3	8·4	5·8	5·2	14·2	6·8	7·9	11·2	11·1	16·9	60·5
October	7·4	7·4	4·4	3·2	3·5	2·4	4·8	3·2	3·1	4·3	4·5	5·2	17·7
November	1·2	·8	·0	·1	·1	·2	·3	0·5	·8	·6	1·3	1·1	3·1
December	·9	·0	·1	·1	·1	·0	·1	·1	·9	·4	·7	·1	·0
Year	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

III.—Bombay.—Mean Monthly Rainfall for 24 Places before 1861.

Place	Vingorla.	Dharwar.	Belgaum.	Sawantwari.	Kaladghi.	Kolapor.	Rutnaghari.	Sholapor.	Sattara.	Malcompet.	Mahabhleshwar.	Purandhar.
Latitude	15° 50′	15° 50′	15° 52′	15° 56′	16° 11′	16° 42′	17° 0′	17° 40′	17° 40′	17° 56′	17° 59′	18° 12′
Longitude	73° 41′	75° 10′	74° 42′	74° 1′	75° 33′	74° 18′	73° 20′	76° 0′	74° 2′	73° 41′	73° 30′	73° 54′
Height	20	2482	2260	...	1750	1797	150	1821	2320	...	4500	4200
No. of Years	4	6	10	2	5	10	9	11	5	6	15	6
Actual Years	1856–59	1844–47 1859–60	1841–47 1858–59	1848–49	1855–59	1850–59	1844–47 1855–59	1850–60	1855–59	1843–47	1829–43	....
January	·0	·0	·0	·0	·1	·4	·0	·1	·6	·4	·0	·1
February	·0	·0	·0	·0	·0	·0	·0	·5	·0	·0	·2	·3
March	·0	·6	·8	·0	·1	·6	·0	·3	·1	1·7	·1	·2
April	·4	1·3	3·3	·0	1·5	1·4	·0	1·9	1·0	2·3	1·3	·5
May	16·7	3·7	3·9	2·2	3·8	3·6	3·6	2·3	3·3	2·2	3·3	5·7
June	29·0	4·6	7·9	32·2	1·6	6·6	31·0	3·7	5·0	48·6	46·5	10·2
July	40·9	7·1	15·0	48·3	2·9	11·9	30·5	4·3	13·1	100·2	92·1	23·0
August	22·2	8·4	8·4	29·2	2·5	5·1	14·9	6·2	7·1	68·3	72·3	16·3
September	6·2	3·0	4·3	16·5	4·1	4·6	9·0	6·8	3·1	33·8	31·3	7·4
October	2·9	2·8	5·2	2·6	6·7	4·5	21·9	4·7	4·7	2·4	4·6	6·5
November	·1	3·2	2·4	·3	·6	·7	1·3	1·4	·6	1·7	2·1	·7
December	·0	·1	·0	·0	·4	·1	·0	·5	·6	·1	·0	1·4
Year	118·	35·	51·	131·	24·	39·	112·	33·	39·	262·	254·	72·

Place	Puna.	Serur.	Bombay.	Nasik.	Dhulia.	Baroda.	Rajkot.	Ahmadabad.	Ahmadnagar.	Kotri.	Mount Aboo.	Haiderabad.
Latitude	18° 30′	18° 50′	18° 53′	20° 0′	20° 51′	22° 16′	22° 18′	23° 0′	23° 24′	23° 54′	24° 45′	25° 30′
Longitude	74° 0′	72° 58′	72° 52′	73° 47′	74° 45′	73° 14′	70° 50′	72° 0′	73° 1′	68° 46′	72° 49′	69° 5′
Height	1800	1752	14	1843	1000	90	450	320	1900	..	4000	99
No. of Years	18	5	34	4	6	17	4	2	10	5	5	2
Actual Years	various 1826–60	1854–58	1817–50	1844–47	1853–58	1837–53	1857–60	1858–59	1844–47 1854–59	1845–49	1855–59	1856–57
January	·4	·1	·0	·0	·0	·0	·0	·0	·1	·0	·8	·6
February	·0	·3	·0	·0	·8	·0	·0	·0	·1	·0	·2	·5
March	·7	·4	·0	·0	·6	·0	·0	·0	·4	·1	·0	·0
April	·8	·9	·0	·0	·5	·0	·0	·0	·4	·0	·3	·6
May	3·8	2·9	·0	·0	2·3	·8	·0	·0	3·0	·4	·3	·0
June	6·9	3·7	22·1	6·8	15·3	6·2	4·2	1·8	4·6	1·3	10·3	·0
July	11·1	3·0	24·9	6·7	21·4	14·2	8·3	15·9	2·5	·0	25·9	1·9
August	6·5	1·5	16·8	3·4	9·5	7·4	8·9	7·6	3·3	·0	17·0	14·8
September	4·8	3·9	11·1	2·9	20·9	5·2	5·2	6·0	6·5	·0	9·9	2·1
October	4·5	3·7	1·3	3·3	13·3	·8	·0	·0	3·2	·0	·0	·0
November	3·9	·7	·0	1·5	·0	·0	·0	·0	1·1	·0	·0	·0
December	·2	·0	·0	·0	·1	·0	·0	·0	·4	·0	·0	·0
Year	44·	21·	76·	29·	84·	35·	27·	31·	26·	2·	65·	21·

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.

Place	Special Observatories.					Ahmadabad.	Ahmadnagar.	Ahmodi.	Alibagh.	Amalner.
	Belgaum.	Puna.	Bombay.	Karachi.	Deesa.					
Latitude	15° 52′	18° 30′	18° 53′	24° 51′	25° 14′	23° 0′	23° 34′	21° 59′	18° 40′	28° 3′
Longitude	74° 42′	74° 0′	72° 52′	67° 2′	72° 5′	72° 0′	73° 1′	72° 53′	72° 58′	75° 1′
Height	2260	1800	14	27	400	320	1900	...	...	...
No. of Years	10	10	10	10	10	7	8	6	7	8
January	·00	·01	·04	·85	·12	·0	·4	·0	·0	·0
February	·01	·10	·01	·19	·18	·0	·0	·0	·0	·0
March	·51	·34	·00	·26	·25	·0	·3	·0	·0	·0
April	1·92	·44	·01	·01	·00	·0	·5	·0	·0	·0
May	1·83	1·01	·08	·15	·03	·0	·9	0	·0	·0
June	12·11	7·45	17·24	·17	2·82	3·9	5·2	5·9	18·6	4·0
July	14·82	5·74	24·06	2·72	8·13	7·8	2·6	7·1	24·4	8·4
August	12·79	5·95	17·94	2·74	10·31	13·6	4·6	13·1	16·9	6·7
September	2·01	4·29	9·47	1·03	2·40	2·4	5·6	2·6	9·2	4·5
October	4·56	6·77	2·42	·19	1·63	·0	3·8	·6	·3	1·0
November	·28	·48	·23	·05	·07	·0	·5	·0	·0	·0
December	·62	·35	·01	·29	·00	·0	·0	·0	·0	·0
Mean Year	51·44	32·99	71·53	8·65	25·93	27	24	29	69	25
Min. Year	42·98	22·34	45·61	2·41	15·93	18	17	15	57	15
Max. Year	64·69	57·79	91·66	28·45	38·94	46	37	42	78	57

Place	Anand.	Ankola.	Antry.	Badami.	Baglan.	Bagalkot.	Bejaiwadi.	Balsar.	Barawatti.	Bardoli.	Baroda.
Latitude	...	...	...	15° 55′	...	16° 12′	...	20° 36′	18° 10′	21° 4′	22° 16′
Longitude	...	...	...	75° 45′	...	75° 25′	...	72° 58′	74° 30′	73° 10′	73° 14′
Height	...	...	...	...	...	...	...	48	...	...	90
No. of Years	2	5	8	3	8	3	4	8	2	8	7
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	·1	·0	·0	·0	·0	·0	·0	·0	·2
June	5·5	5·2	3·6	2·2	3·6	3·1	1·8	17·4	4·2	8·0	6·3
July	7·3	5·3	2·9	3·5	3·9	2·9	4·4	23·3	·8	12·0	8·1
August	10·9	5·1	4·0	1·6	3·4	1·7	4·0	18·5	3·7	22·7	13·8
September	·0	3·3	2·3	5·5	2·8	5·8	3·4	9·0	2·2	5·6	2·4
October	1·2	3·5	4·3	7·1	1·4	4·7	3·6	·1	·0	1·0	0·5
November	·0	·2	·1	·0	·1	·0	·9	·0	·0	·0	·1
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	24	23	17	20	15	18	18	68	11	49	32
Min. Year	16	15	18	14	9	15	12	47	8	29	21
Max. Year	34	25	23	25	25	25	29	87	14	62	45

N.B.—The Latitudes and Longitudes given for these places in the Bombay Presidency have been taken from

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Barsi.	Bassein.	Bedi.	Belgaum.	Bejapur.	Belzi.	Bhargaum.	Bhewandi.	Bhaunagar.	Brahmanwade.	Broach.	Borsad.
Latitude	18° 16′	19° 20′	...	15° 52′	16° 50′	...	...	19° 19′	21° 45′	...	21° 42′	...
Longitude	75° 46′	72° 52′	...	74° 42′	75° 40′	...	...	73° 9′	72° 10′	...	73° 2′	...
Height	1682	11	..	2260	...	...	...	...	...	...	56	...
No. of Years	3	7	7	5	2	3	7	3	8	5	7	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	1·0	1·9	·0	1·0	·0	·0	·0	·0	·0	·0
June	5·0	16·2	11·8	10·6	5·4	3·0	4·1	13·7	5·3	4·5	7·0	4·5
July	3·3	20·3	13·2	9·6	1·2	1·3	6 7	24·2	4·8	4·5	11·8	9·0
August	4·4	17·3	8·1	7·1	4·0	2·0	7·2	16·7	10·4	7·2	11·4	14·7
September	8·4	8·7	1·7	2·6	8·2	5·4	4·4	7·5	1·9	2·8	3·8	3·4
October	3·0	·7	1·3	1·7	2·8	1·8	·9	2·7	1·1	·0	·4	·1
November	·1	·0	·1	·0	·0	·1	·0	·2	·0	·0	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	24	63	36	34	22	13	23	65	24	19	34	32
Min. Year	19	54	27	32	17	.	19	7	9	17	21	21
Max. Year	27	78	57	50	43	18	34	96	42	22	51	50

Place	Byculla.	Carwar.	Callian.	Callian.	Chalisgaum.	Chandor.	Chikli.	Chikodi.	Chiplon.	Chopra.	Colaba.
Latitude	18° 57′	14° 50′	...	...	20° 40′	20° 20′	22° 19′	...	17° 30′	21° 14′	18° 53′
Longitude	72° 52′	74° 15′	...	...	75° 5′	74° 14′	74° 41′	...	73° 36′	75° 27′	72° 52′
Height	13	...	...	26	1138	1806	...	...	...	...	14
No. of Years	7	8	3	3	8	8	8	7	4	8	7
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·4	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	1·1	·0	·0	·0	·1	·0	·6	·2	·0	·2
June	20·7	34·4	16·7	16·7	5·2	5·9	16·7	5·2	37·3	3·6	15·5
July	26·2	36·9	31·2	31·2	5·4	4·7	26·9	4·1	42·4	6·8	24·4
August	20·4	23·7	27·9	27·9	5·7	5·4	20·9	3·4	36·9	4·9	18·1
September	8·7	9·0	6·4	6·4	5·6	3·8	6·9	2·7	9·5	2·1	8·7
October	·7	3·4	3·2	3·2	·9	2·8	5·9	1·4	4 3	·3	·8
November	·0	1·1	·0	·0	·0	·2	·0	·0	·3	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	77	110	85	85	23	21	77	17	131	20	68
Min. Year	56	80	81	81	13	14	13	53	119	14	45
Max. Year	80	145	91	91	27	28	104	26	145	32	81

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Compta.	Dahnu.	Dahwudi.	Dapuli.	Dewarhobli.	Dohud.	Dumbul.	Dindor.	Dhalka.	Dharwar.	Dhunduka.
Latitude	14 25	19 55	..	17 48	...	...	15 18	20 1	...	15 28	...
Longitude	74 20	72 40	..	73 16	...	...	75 50	73 50	...	75 54	...
Height	...	...	..	...	...	...	...	...	...	...	...
No. of Years	8	7	7	8	7	8	7	8	6	4	6
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·5	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·5	·0	·0	·0	·0	·0	·0	·0	·0	3·4	·0
June	31·8	15·5	3·6	30·7	5·7	5·5	2·7	4·6	2·5	7·2	3·6
July	44·7	19·3	1·4	39·4	6·7	10·9	1·1	7·6	7·3	4·8	5·7
August	23·5	19·8	2·1	31·7	5·4	8·2	1·8	8·9	11·1	4·8	11·3
September	7·6	7·6	3·6	11·3	2·1	2·8	2·6	3·2	1·7	2·6	2·4
October	3·7	·8	3·8	3·3	1·3	·7	·9	3·7	·0	2·5	·1
November	·3	·0	·0	·0	·0	·0	·0	·2	·0	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	113	63	15	117	21	28	9	27	23	25	23
Min. Year	11	43	9	91	16	24	6	21	13	18	11
Max. Year	163	79	20	145	26	35	18	34	31	31	33

Place	Dhulia.	Egutpura.	Erandol.	Esplanade.	Fort St. George.	Ghodnudi.	Godal.	Godra.	Gogo.'	Gohagar.	Gokak.	Godaj.
Latitude	20 54	19 43	20 56	18 57	18 57	...	...	22 45	21 39	...	16 10	15 26
Longitude	74 45	73 34	75 19	72 52	72 52	...	...	73 36	72 15	...	74 53	75 43
Height	...	...	...	14	14	...	...	...	...	...	...	...
No. of Years	8	8	7	7	7	2	7	5	7	8	7	7
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·5	·0
June	4·0	23·9	5·1	19·6	19·9	4·6	4·7	5·9	4·9	22·2	3·4	3·0
July	3·5	40·3	6·0	25·4	26·2	1·0	3·9	7·5	6·1	31·7	1·3	1·4
August	4·1	42·7	7·4	19·4	19·3	2·5	6·1	18·9	6·5	17·6	1·9	3·3
September	2·3	13·6	3·1	9·4	8·8	3·2	1·3	8·3	2·5	8·1	2·2	2·7
October	·7	4·9	·5	·6	·6	·6	·4	·1	·0	2·7	1·8	1·3
November	·1	·0	·0	·0	·0	·1	·1	·0	·0	·7	·1	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	15	126	22	74	75	12	16	35	20	83	11	12
Min. Year	9	105	16	57	62	10	11	28	15	56	4	7
Max. Year	27	163	34	84	87	14	23	42	30	141	18	21

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Gokal.	Halial.	Halole.	Hangal.	Hansot.	Hirvat.	Honori.	Hobli.	Hungund.	Indapor.	Indi.	Jamner.	Jhaloda.	Kadkalay.	Kaira.	Kalole.	Karjat.	Karjat.	Kaliwadi.	Karatgaum.	Kharepatan.	Khed.	Khed.
Latitude	14° 50′	15° 20′	...	14° 50′	21° 32′	...	14° 10′	15° 20′	16° 3′	18° 10′	17° 10′	20° 48′	23° 5′	...	22° 45′	...	...	...	...	...	...	18° 51′	18° 23′
Longitude	75° 42′	74° 40′	...	75° 5′	72° 50′	...	74° 25′	75° 13′	76° 9′	75° 5′	75° 57′	75° 44′	74° 3′	...	72° 41′	...	...	...	...	...	...	73° 55′	73° 53′
Height	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...	...
No. of Years	6	8	3	7	4	5	7	7	3	7	3	3	3	7	3	3	8	7	8	8	8	7	6
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	1·2	·0	·0	·0	·0	·6	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	1·3	·0	·0	·0	·0	1·7	·0	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0
June	2·1	14·9	3·3	6·1	2·4	4·2	34·3	4·7	2·4	3·3	·3	4·2	1·8	14·4	4·1	4·3	3·1	21·6	11·5	3·6	34·8	5·1	33·3
July	1·8	20·7	8·5	8·6	9·6	3·6	32·2	3·9	4·2	1·2	6·1	7·0	9·8	22·5	9·5	10·1	1·8	39·3	18·3	2·0	43·1	3·5	41·7
August	2·4	19·2	20·0	6·0	16·7	4·5	26·0	3·3	2·2	1·0	3·1	6·8	13·4	23·3	11·1	17·3	2·1	39·7	15·4	3·9	29·2	4·6	30·8
September	3·0	4·2	2·0	2·0	2·1	1·8	15·1	2·2	6·4	1·8	3·9	3·3	2·1	3·9	2·1	2·1	5·4	10·7	4·4	3·8	10·9	2·3	9·1
October	1·8	7·3	·1	1·3	1·2	·0	3·2	1·1	3·6	·0	8·0	·4	·8	·0	·1	·8	1·3	1·3	1·1	3·1	4·5	·4	3·9
November	·0	1·0	·0	·0	·0	·0	·6	·0	·0	·0	3·3	·0	·0	·0	·0	·0	·1	·0	·0	·5	·4	·0	·4
December	·0	·2	·0	·0	·0	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	11	70	34	24	32	14	114	15	19	7	25	22	28	64	27	35	14	113	51	17	123	16	119
Min. Year	7	28	28	22	23	7	8	8	16	3	16	17	26	45	16	27	11	85	26	12	102	11	100
Max. Year	18	132	37	29	48	21	175	22	24	11	29	25	30	93	33	39	21	136	70	28	143	26	133

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Khandalla.	Karadh.	Kurnalla.	Koiladghi.	Kunkowli.	Kupperwang.	Kurjugi.	Kolhapur.	Koregaum.	Lanza.	Lohara.
Latitude	18° 48′	17° 15′	18° 25′	16° 10′	...	..	..	16° 41′	17° 41′	...	20° 42′
Longitude	73° 26′	74° 10′	75° 15′	75° 25′	...	..	..	74° 10′	74° 15′	...	75° 32′
Height	1793′	..	...	...	...	...	...	...	...	...	...
No. of Years	2	8	7	4	7	8	7	8	8	3	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·2	·0	·0	·0	·1	·0	·0	·0
April	·0	·0	·0	·2	·0	·0	·0	1·8	·0	·0	·0
May	·1	·0	1	1·1	·2	·0	·0	2·1	·0	·3	·0
June	7·4	6·5	3·9	3·2	31·0	4·9	3·1	8·7	6·6	26·5	5·5
July	4·9	6·5	1·9	1·4	41·7	8·8	3·5	9·1	6·1	47·3	9·1
August	2·4	5·9	2·3	3·1	26·7	10·8	3·1	7·9	6·1	25·0	8·6
September	1·2	1·9	4·3	3·2	10·2	3·1	2·0	2·9	2·4	11·1	5·0
October	5·1	5·5	2·1	3·2	4·0	·1	2·0	5·1	4·0	4·2	·5
November	·1	·2	1·0	·1	·2	·0	·0	·3	·2	·6	·0
December	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0
Mean Year	21	26	15	16	114	28	14	38	25	115	29
Min. Year	15	20	4	7	100	15	8	29	22	100	19
Max. Year	27	32	30	27	140	52	20	46	38	128	37

Place	Mahad.	Mahim.	Malshiras.	Malcolmpet.	Malwan.	Malligaum.	Malunda.	Mangaon.	Mandvee.	Manikwara.	Matur.	Medhai.
Latitude	...	19° 40′	...	17° 56′	...	20° 32′	...	15° 20′	22° 51′	21° 20′	...	...
Longitude	...	72° 47′	...	73° 48′	...	74° 30′	...	73° 55′	69° 26′	70° 40′	...	...
Height	...	13	...	...	...	...	...	...	...	...	...	...
No. of Years	7	7	7	8	3	8	6	2	8	2	5	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0
May	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0	·2
June	27·9	16·1	2·4	16·7	18·7	4·2	4·5	30·9	9·4	2·8	5·2	16·9
July	43·6	22·7	1·5	95·1	32·1	3·2	11·1	36·6	1,·1	6·6	9·9	21·0
August	37·7	20·2	2·4	86·1	11·7	3·9	9·9	30·6	17·8	8·0	6·8	20·0
September	12·6	7·1	4·9	23·9	5·6	5·7	2·0	11·4	3·9	3·1	5·1	2·7
October	1·1	1·8	3·9	6·6	4·8	1·4	·0	8·2	1·4	2·1	·3	3·6
November	·0	·0	·1	·4	·2	·0	·0	·0	·0	·1	·0	·3
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	123	68	15	260	73	18	28	118	48	22	27	65
Min. Year	98	59	9	214	58	12	14	97	36	22	25	41
Max. Year	171	81	28	312	86	26	42	138	57	22	32	81

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Mehmunabad.	Minrikot.	Moolshi.	Mokhada.	Mohole.	Murbad.	Mundnagar.	Mudibahal.	Nariad.	Nasik.	Nasirabad.	Niphar.
Latitude	18° 42′	15° 16′	...	...	17° 48′	19° 17′	...	16° 17′	...	20° 0′	20° 59′	20° 5′
Longitude	76° 18′	78° 8′	..	...	75° 42′	73° 30′	...	76° 5′	...	73° 47′	75° 37′	73° 45′
Height	5	7	7	7	1493	...	...	...	...	1843	...	...
No. of Years	...	...	...	...	4	7	7	5	8	8	8	8
January	·0	·0	0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
June	1·9	5·8	12·2	17·8	4·2	15·2	33·6	2·8	4·7	4·5	4·4	3·1
July	5·5	6·3	15·8	30·7	3·7	26·7	46·0	2·1	10·7	6·1	8·5	2·4
August	11·9	4·1	14·6	31·2	3·3	28·8	32·7	4·7	11·3	6·3	7·2	4·5
September	1·4	2·3	3·2	9·6	2·6	8·1	11·6	3·6	1·9	3·6	4·8	2·4
October	·0	1·8	·1	1·6	5·3	·9	3·9	2·4	·1	3·9	·4	3·7
November	·0	·0	·0	·0	·1	·0	·7	·5	·0	·3	·0	·1
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	21	20	46	91	19	71	128	16	29	25	25	16
Min. Year	15	9	27	74	13	65	109	8	17	16	17	11
Max. Year	31	26	55	108	30	92	136	23	40	32	36	20

Place	Newasa.	Nundoorbar.	Nuppur.	Nurgoond.	Nuvulgoond.	Oomburgaon.	Ooljar.	Oorun.	Pabal.	Padshapur.	Palitana.
Latitude	...	21° 22′	...	15° 43′	...	...	...	...	..	16° 5′	21° 30′
Longitude	...	74° 12′	...	75° 27′	...	...	...	...	..	74° 46′	71° 40′
Height	...	...	...	...	...	...	...	...	...	...	...
No. of Years	8	8	3	7	7	2	8	2	5	5	8
January	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
June	3·4	4·3	5·5	3·2	4·4	5·7	6·5	10·2	4·4	10·5	·0
July	3·0	6·1	10·6	1·6	2·8	20·8	8·5	32·6	2·6	12·1	12·4
August	2·9	8·0	5·5	2·5	3·3	22·3	9·9	20·6	3·9	9·1	13·5
September	4·7	2·2	2·3	2·5	4·0	5·1	2·6	9·5	3·0	1·8	2·5
October	1·5	·8	·0	1·7	1·3	3·0	·5	3·1	·0	·0	1·1
November	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	16	21	24	12	16	57	28	76	14	34	30
Min. Year	6	13	19	6	6	40	16	73	7	30	20
Max. Year	23	26	31	19	30	74	49	80	18	40	42

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—*Continued.*

Place	Pamkher.	Panwell.	Pandarpur.	Paragud.	Pardee.	Parneir.	Parentay.	Patun.	Patus.	Peint.	Peint.	Pimpalner.
Latitude	...	18 58	17 40	...	...	19 0	...	...	18 28	...	20 17	20 55
Longitude	...	73 12	75 24	...	...	74 29	...	...	74 25	...	73 31	74 4
Height	...	...	...	...	...	...	...	...	...	...	...	...
No. of Years	8	7	4	7	8	8	6	7	7	7	8	8
January	·2	·0	·0	·0	·0	·1	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·0	·0	·1	·0	·1	·0	·3	·1	·0	·1	·0
June	6·6	22·8	3·7	4·5	16·2	5·1	3·7	15·1	4·2	21·5	14·1	3·6
July	3·4	36·4	2·2	2·5	24·6	2·4	7·8	16·7	·8	32·8	35·8	5·7
August	5·5	31·8	3·3	2·2	21·9	4·0	14·8	18·6	2·0	23·2	37·0	6·7
September	6·9	10·6	4·2	2·8	8·2	4·3	2·9	3·1	3·7	7·7	9·1	2·1
October	3·4	1·3	3·4	1·7	1·3	4·6	·0	3·4	·1	·3	4·1	1·3
November	·2	·0	·0	·0	·0	·3	·0	·2	·0	·0	·0	·0
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	26	103	17	15	72	21	29	55	11	85	100	19
Min. Year	16	90	7	6	48	14	24	42	7	68	81	13
Max. Year	38	112	26	27	95	28	34	84	20	92	121	22

Place	Puna.	Rahuri.	Rajkot.	Ranibedor.	Raveri.	Rewarkanta.	Rohey.	Rojapur.	Rutchal.	Rutnagheri.	Sadra.
Latitude	18 31	19 23	22 18	14 37	21 14	...	...	...	...	16 58	...
Longitude	73 53	74 40	70 50	75 41	76 14	...	...	...	...	73 15	...
Height	1819	...	...	...	...	...	...	...	...	...	...
No. of Years	8	8	8	7	7	5	7	4	7	8	8
January	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·1	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
April	·2	·0	·0	·0	·0	·0	·0	·0	·0	·3	·0
May	·6	·1	·1	·0	·0	·0	·0	·2	·0	·3	·1
June	5·2	4·8	6·8	·5	5·4	4·3	25·6	31·2	4·5	30·6	4·6
July	4·6	2·2	8·9	2·1	6·0	9·7	36·2	40·9	4·2	35·2	10·5
August	5·4	3·2	10·1	2·1	7·7	14·8	29·9	34·3	3·5	23·2	14·1
September	3·6	5·8	3·1	2·0	4·7	2·2	9·8	7·9	8·6	11·4	5·3
October	7·1	2·9	1·5	1·8	·6	·1	1·0	3·6	1·7	2·6	·7
November	·2	·2	·0	·1	·2	·0	·0	·4	·0	·6	·2
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·2
Mean Year	27	19	31	12	24	31	104	119	22	104	34
Min. Year	19	14	20	7	15	25	89	106	11	90	22
Max. Year	47	24	44	21	35	41	116	131	67	123	55

III.—Bombay.—*Mean Monthly Rainfall of 205 Places between 1860 and 1869 inclusive.—Continued.*

Place	Sangola.	Sattara.	Sawurgaon.	Sawantwari.	Sanand.	Sasoor.	Shiraloy.	Shahapur.	Sheogaum.	Shrigonda.	Shegaum.	Shindgi.
Latitude	17° 26′	17° 45′	20° 7′	15° 56′	...	18° 20′	...	...	...	...	...	...
Longitude	75° 16′	74° 4′	74° 25′	74° 1′	...	74° 3′	...	...	...	...	...	...
Height	...	...	...	...	...	...	...	...	...	...	...	...
No. of Years	4	2	8	8	2	7	3	7	8	2	7	5
January	·0	·0	·1	·0	·0	·0	·0	·0	.2	·2	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·1	·0	·0	·0
April	·0	·0	·0	·5	·0	·0	·0	·0	·0	·0	·0	·0
May	·0	·1	·0	·4	·0	·0	·4	·0	·0	·2	.0	·0
June	3·2	9·5	3·9	40·5	2·4	2·1	8·4	22·8	3·6	7·3	4·8	·1
July	2·1	10·2	3·0	46·9	8·3	1·0	6·5	39·7	4·1	2·1	4·5	4·9
August	2·8	10·8	4·4	33·6	17·6	1·8	7·6	32·3	5·5	2·4	3·2	3·1
September	3·8	2·1	2·8	11·0	5·0	1·1	1·5	14·2	5·9	4·0	1·2	3·7
October	1·6	4·4	2·7	4·8	·2	·2	6·4	1·8	1·6	5·4	1·9	5·3
November	·0	·6	·6	·6	·0	0	·0	·0	·0	·6	·0	3·3
December	·0	·0	·0	·6	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	14	38	18	139	33	6	31	111	21	22	16	20
Min. Year	7	29	10	97	23	2	27	96	14	16	12	10
Max. Year	22	48	22	176	44	17	35	150	27	28	21	27

Place	Sholapor.	Sinnar.	Sirey.	Soopa.	Soopa.	Sonpuree.	Siddopur.	Sungumeshwar.	Surat.	Sungamner.	Sultanpur.
Latitude	17° 40′	19° 51′	...	18° 20′	15° 16′	...	14° 21′	17° 8′	21° 10′	19° 35′	21° 45′
Longitude	76° 0′	74° 0′	...	74° 26′	74° 35′	...	74° 58′	73° 35′	72° 52′	74° 14′	74° 20′
Height	1495	...	...	...	1250	...	...	...	59	...	...
No. of Years	8	8	8	5	8	2	7	4	8	8	8
January	·2	·0	·2	·0	·0	·0	·0	·0	·0	·0	·0
February	·2	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·3	·0	·7	·0	·1	·0	·9	·0	·0	·0	·0
April	·5	·0	1·1	1·1	·0	·0	1·1	·0	·0	·0	·0
May	·8	·0	1·6	·0	·9	·0	1·1	·1	·0	·3	·0
June	3·8	4·0	24·1	1·8	14·4	2·4	27·4	31·9	9·7	4·4	4·1
July	4·5	3·4	35·3	·9	40·9	10·6	39·5	50·9	9·9	2·0	6·3
August	5·5	4·8	27·0	1·0	30·8	16·1	27·0	41·2	12·2	3·0	8·8
September	5·5	2·6	16·1	1·9	7·6	5·0	7·5	9·3	3·6	3·5	2·4
October	2·5	5·5	6·5	·0	5·6	·2	7·0	3·8	1·7	4·4	1·4
November	·4	·4	·5	·0	·8	·0	·3	·7	·1	·7	·0
December	·0	·0	·2	·0	·0	·0	·0	·0	·0	·1	·0
Mean Year	24	21	113	5	101	34	112	138	37	18	23
Min. Year	19	16	89	4	74	26	27	125	20	11	17
Max. Year	28	26	189	8	146	43	144	152	52	29	29

III.—Bombay.—Mean Monthly Rainfall of 205 Places between 1860 and 1866 inclusive.—*Continued.*

Place	Sumpgure.	Sowda.	Thalnar.	Tharu.	Tanna.	Tasgaum.	Tooneer.	Tumboosur.	Unklesur.	Uthnee.	Veerangaon.	Vinchoor.
Latitude	...	21° 8′	21° 15′	...	..	17° 2′	..	...	...	...	23° 71′	...
Longitude	...	76° 0′	75° 6′	..	...	74° 40′	...	. .	...	...	72° 0′	...
Height	..	708	...	...	...	...	...	...	65	...	...	...
No. of Years	7	8	8	8	7	6	2	7	6	7	6	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·2
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·3
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1
May	·8	·0	0	·0	·1	·2	·0	·0	·0	·0	·0	·5
June	5·7	5·1	4·4	4·9	23·0	4·1	4·5	4·0	5·0	3·5	4·2	3·1
July	3·9	6·9	6·6	9·2	31·7	2·4	5·0	8·6	12·3	1·1	6·0	3·5
August	3·2	5·9	6·0	10·9	26·6	3·5	7·8	8·6	14·1	2·7	9·2	4·1
September	1·6	3·4	2·2	3·2	9·5	3·3	2·0	5·6	2·9	3·2	3·4	2·1
October	1·2	·5	·5	·2	·0	3·5	1·0	·3	·4	1·1	·0	3·1
November	·0	·0	·1	·0	·0	·1	·2	·0	·0	·0	·0	·2
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
Mean Year	16	28	20	28	91	17	21	27	35	12	23	17
Min. Year	7	14	10	18	76	...	16	13	21	6	14	12
Max. Year	24	30	31	46	101	...	25	36	45	20	29	24

Place	Vingorla.	Virdale.	Vitey.	Wadoon.	Wadwan.	Waee.	Wagra.	Walore.	Warra.	Wurrungaum.	Yellapo.
Latitude	15° 50′	...	...	...	22° 40′	19° 50′	...	...	...	...	14° 58′
Longitude	73° 41′	...	...	...	71° 30′	74° 12′	...	...	...	...	74° 46′
Height	...	...	...	...	...	...	...	...	...	245	...
No. of Years	3	8	8	2	2	8	7	4	2	7	8
January	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
February	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0
March	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	1·1
April	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·6
May	1·3	·0	·1	·0	·0	·0	·0	·0	·0	·0	11·2
June	39·2	4·3	3·8	4·1	2·0	7·3	3·7	6·5	15·1	4·4	20·6
July	39·4	4·6	3·1	2·2	3·3	6·6	8·5	16·2	31·8	7·6	29·8
August	0·3	5·6	4·8	3·8	13·3	6·0	9·3	25·8	29·9	6·3	22·4
September	7·3	2·2	3·0	3·5	4·8	2·3	3·2	5·1	6·8	4·1	4·6
October	3·3	·7	6·0	3·0	1·2	4·9	·3	3·1	4·2	·5	4·6
November	·3	·1	·1	·1	·0	·2	·0	·0	·0	·0	·6
December	·0	·0	·0	·0	·0	·0	·0	·0	·0	·0	·1
Mean Year	111	17	21	17	25	27	25	57	88	23	95
Min. Year	108	12	11	9	11	18	10	41	84	17	69
Max. Year	116	23	38	30	38	33	32	73	92	38	190

IV.—North-West Provinces and V. Panjab.—Mean Monthly Rainfall before 1861.

North-West Provinces.

Place	Benares.	Ghazipur.	Azimgarh.	Hamirpur.	Kherwara.	Etawah.	Gorakpur.	Fattehgarh.	Agra.	Mirut.	Rurkhi.	Landaur.	Naini Tal.
Latitude	25° 18′	25° 49′	26° 3′	26° 7′	26° 42′	26° 45′	26° 46′	27° 23′	27° 10′	28° 55′	29° 53′	30° 27′	29° 20′
Longitude	83° 3′	80° 48′	83° 13′	79° 47′	79° 12′	79° 3′	83° 18′	79° 30′	78° 5′	77° 40′	77° 57′	78° 10′	79° 31′
Height	270	...	..	..	1200	..	255	600	551	...	880	7000	6463
No. of Years	2	4	1	1	5	1	1	2	1	1	1	10	9
Actual Years	1858–59	1856–59	1851	1851	1854–58	1851	1851	1848–49	1851	1859	1860	1850–59	1846–54
January	1·2	·8	3·8	1·5	·0	2·8	3·6	·9	1·2	·2	·0	·6	4·6
February	·1	1·7	1·1	·8	·0	·6	·2	·0	1·1	·2	·0	·6	2·4
March	·1	·4	·6	·0	·0	·0	·2	·0	·0	1·8	·0	·3	1·7
April	·3	·4	·8	·3	·0	·1	1	·0	·2	2·6	·0	1·9	·9
May	·0	·5	·0	·0	·0	·3	1·4	·0	·7	·0	·5	1·5	2·8
June	6·3	7·4	4·5	6·7	4·0	·9	15·6	2·7	·3	3·6	1·5	11·9	14·7
July	7·1	8·6	8·7	12·6	12·3	11·4	14·1	2·5	9·8	1·5	4·0	31·5	23·4
August	5·6	14·4	3·8	10·5	3·9	9·2	5·0	5·4	10·0	5·9	3·0	32·3	28·8
September	9·7	8·2	9·5	4·8	6·5	6·2	5·9	10·7	4·0	1·4	1·0	10·3	12·6
October	3·8	4·2	7·2	·7	·8	·3	11·6	2·9	·6	·0	·3	·9	2·7
November	·0	·3	·0	·0	·0	·0	·0	·0	·0	·0	·0	·6	·8
December	·0	·4	·0	·0	·0	·1	·0	·3	·0	·8	·0	·3	·2
Year	37	47	40	38	28	32	62	25	28	18	10	93	96

Panjab.

Place	Delhi.	Ambala.	Mian Mir.	Firozpur.	Ludhiana.	Panjar.	Mira.	Rawalpindi.	Peshawar.
Latitude	28° 31′	30° 23′	30° 34′	30° 57′	30° 55′	31° 40′	32° 37′	33° 4′	34° 20′
Longitude	77° 13′	76° 44′	74° 4′	74° 41′	75° 54′	74° 45′	70° 30′	73° 5′	71° 29′
Height	..	1050	1128	710	900	850	...	1700	1056
No. of Years	1	2	3	5	5	3	1	1	6
Actual Years	1851	1851–52	1857–59	1855–59	1857–59	1857–59	1848	1859	1855–60
January	2·6	·7	1·2	1·0	2·5	1·1	·0	·0	1·2
February	·3	·0	1·4	·2	1·1	2·0	·0	·0	1·8
March	2·6	7·1	·4	·6	·1	·7	·0	·0	1·1
April	1·0	·9	·5	·9	·8	5·2	·0	·0	·8
May	·1	2·8	·4	·1	·5	1·2	2·8	·0	·4
June	·4	1·3	1·4	1·1	1·6	8·4	10·9	4·1	·3
July	11·6	7·3	5·5	6·5	2·9	22·7	22·9	4·8	2·1
August	6·5	3·1	3·6	1·3	4·3	10·5	6·2	5·1	3·3
September	·0	·0	1·9	1·9	2·3	4·2	2·3	5·6	1·7
October	·0	·0	·0	·0	·0	·0	2·8	·0	·1
November	·0	2·3	·0	·0	·2	·0	1·9	·0	·3
December	·0	·1	·0	·0	·2	·6	·0	·0	·6
Year	25	16	16	14	17	57	50	18	14

IV.—North-West Provinces and Oudh.—Mean Monthly Rainfall between 1867 and 1872 inclusive.

Place	Chakrata.	Ranikhet.	Dehra.	Rurkhi.	Mirat.	Bareli.	Agra.	Fattahgarh.	Lakhnau.	Allahabad.	Banaras.
Latitude	30 40	29 38	30 20	29 53	29 41	28 21	27 0	27 23	26 50	25 26	25 20
Longitude	77 55	79 29	78 8	77 54	77 41	79 27	78 0	79 48	81 0	81 52	83 0
Height	7056	6143	2232	880	739	570	551	503	364	289	263
No. of Years	4	2	5	5	5	5	5	5	5	5	5
Actual Years	1869–72	1871–72	1868–72	1868–72	1868–72	1868–72	1868–72	1868–72	1868–72	1868–72	1868–72
January	·6	2·4	1·9	2·2	1·7	1·5	·9	·9	·8	·4	·8
February	1·7	3·7	3·0	2·3	1·2	·7	·–	·3	·4	·0	·3
March	6·7	·2	3·4	1·7	1·9	1·6	1·3	·7	·8	·6	·5
April	1·3	1·7	·9	·4	·8	·5	·4	·2	·4	·7	·6
May	2·1	3·6	1·4	1·1	1·6	1·4	1·3	·9	1·5	·4	·4
June	8·6	9·6	10·3	6·0	4·9	8·4	2·7	3·7	5·2	8·0	6·2
July	17·1	10·9	28·7	13·9	8·9	13·1	8·4	12·4	14·5	14·7	14·0
August	12·1	12·8	23·9	11·0	5·3	6·1	5·7	6·9	11·1	11·6	8·7
September	6·2	6·3	13·4	5·1	3·0	8·9	4·3	4·8	12·4	11·8	8·1
October	·7	·0	1·3	·9	·0	2·2	·0	·0	3·2	12·9	5·1
November	·0	·3	·0	·0	·0	·0	·0	·0	·0	·0	·0
December	·1	1·5	·6	·5	·9	1·0	·6	1·2	2·8	1·0	·0
Year	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Place	Gorakpur.	Jhansi.	Ajmir.	Morar.	Nagode.	Naugong.	Caunpur.	Sitapur.	Fyzabad.	Naini Tal.
Latitude	26 46	25 27	26 27	26 20	24 35	...	26 37	...	26 48	29 23
Longitude	83 18	78 37	74 40	78 3	80 39	...	80 20	...	82 10	79 31
Height	255	940	1800	800	1100	...	...	...	...	6409
No. of Years	5	5	5	1	3	1	1	2	2	5
Actual Years	1868–72	1868–72	1868–72	1869	1867–69	1869	1866	1868–69	1868–69	1865–69
January	1·1	2·0	·1	·3	·9	·1	·0	·8	·4	6·5
February	·6	·3	·1	·1	·5	·2	·8	·7	·5	5·1
March	·2	·5	·4	1·0	·6	·5	·0	·4	·6	5·5
April	·2	·1	·2	·1	·2	·3	·0	·7	·0	2·4
May	2·4	·4	·6	·2	·3	·0	·0	·9	1·2	2·6
June	4·4	3·2	2·8	·1	11·1	·6	3·2	1·5	4·1	15·3
July	16·5	13·3	6·6	11·0	13·8	33·2	10·9	6·0	14·1	19·3
August	13·2	8·8	6·5	3·5	7·7	6·9	8·9	6·5	3·3	22·3
September	11·7	5·2	4·5	8·0	12·8	8·9	4·1	3·9	12·8	9·2
October	6·8	1·3	·1	4·8	6·0	6·2	·0	2·5	6·3	4·4
November	·0	·0	·2	·0	·0	·0	·0	·0	·0	·0
December	·6	·2	·3	1·4	·4	1·0	·0	·5	·2	1·6
Year	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

V.—PANJAB.—Mean Monthly Rainfall for 6 Years, 1867–72, of 32 places.

Place	Delhi.	Gurgaon.	Karnal.	Hissar.	Rohtak.	Sirsa.	Ambala.	Ludiana.	Simla.	Jhallandar.	Hoshyarpur.	Kangra.
Latitude	28° 31′	28° 28′	29° 42′	29° 16′	28° 53′	..	30° 13′	30° 55′	31° 8′	31° 20′	31° 35′	32° 8′
Longitude	77° 13′	76° 58′	76° 55′	75° 45′	76° 29′	.	76° 44′	75° 54′	77° 8′	75° 26′	75° 50′	76° 15′
Height				..	...	..	1050	900		.		.
No. of Years	6	6	6	6	6	6	6	6	6	6	6	6
January	·8	1·9	1·0	·5	·2	·5	·8	1·0	1·0	·9	1·0	2·7
February	·5	·3	1·1	·4	·6	·5	2·0	1·0	1·4	1·3	1·4	5·6
March	1·2	·6	1·6	·7	·8	·7	1·5	2·0	4·0	1·4	1·8	4·9
April	·5	·3	·4	·2	·4	·3	1·2	·7	2·7	·3	·5	1·7
May	·9	·8	·7	·3	·6	·5	1·0	·8	3·0	1·7	1·2	2·2
June	2·5	2·2	4·8	2·8	1·7	3·1	5·2	2·6	11·2	2·6	3·9	14·4
July	6·8	7·0	7·9	5·2	6·4	2·4	11·4	8·5	17·2	9·0	9·2	40·4
August	6·8	5·6	5·3	4·4	3·9	2·9	8·6	5·9	15·6	7·6	8·5	29·6
September	2·9	4·0	2·9	1·6	2·6	1·8	3·8	3·5	5·7	2·8	4·3	11·5
October	·3	·0	·1	·0	·5	·0	·2	·2	·3	·0	·0	·6
November	0	·0	·0	·0	0	·0	·0	·0	·0	·0	·0	·0
December	·7	·5	·4	·6	·8	·6	·1	3	·4	4	·7	1·1
Mean Year	24	23	26	17	19	13	36	27	63	28	33	115
Min. Year	8	12	11	7	8	8	18	16	51	16	25	67
Max. Year	33	30	38	31	28	20	51	46	74	45	43	191

Place	Amritsar.	Sialkot.	Gurdaspur.	Lahor.	Firozpor.	Gujranwala.	Rawalpindi.	Jhelam.	Guzrat.	Shahpur.	Multan.	Jhang.
Latitude	31° 40′	32° 29′	.	31° 34′	30° 57′	32° 10′	33° 4′	32° 58′	.	..	30° 12′	.
Longitude	74° 50′	74° 35′		74° 21′	74° 41′	74° 10′	73° 5′	73° 42′	.	..	71° 25′	...
Height		900		1000	720	.	1700	.	..	...	.	..
No. of Years	6	6	6	6	6	6	6	6	6	6	6	6
January	1·0	1·3	1·0	·5	·5	1·0	1·9	·7	·9	·1	·3	·3
February	1·2	1·9	2·9	1·0	1·0	·8	1·9	1·4	1·6	·6	·1	·5
March	1·2	2·9	2·6	1·2	1·6	2·0	3·0	1·6	4·1	1·0	·9	2·0
April	·4	1·8	·5	·4	·9	1·1	1·9	·8	1·2	·5	1·1	7
May	·7	1·1	1·2	1·1	·7	·6	·9	·4	·5	·7	·1	·3
June	2·2	3·7	3·3	·9	1·9	1·9	2·0	2·0	3·5	2·0	·1	·6
July	5·3	12·6	7·9	4·6	6·3	6·2	8·1	4·9	9·5	2·7	1·5	3·0
August	4·7	8·9	7·1	2·4	4·6	5·5	4·9	3·5	4·9	3·1	·6	1·9
September	2·9	2·4	3·9	2·4	2·0	2·3	2·9	1·4	2·0	·9	·5	·9
October	·2	·1	·1	·6	·2	·1	·2	·1	·2	·0	·0	·0
November	·0	·0	·6	·0	·0	·0	·0	·0	·0	·0	·0	·0
December	·5	·4	·4	·4	·4	·5	·7	·4	·4	·4	·3	·3
Mean Year	20	37	31	15	20	22	28	17	29	12	6	11
Min. Year	14	23	23	9	11	12	21	13	21	5	2	3
Max. Year	28	47	43	20	35	32	31	20	32	25	16	14

V.—Panjab *continued.*

Place	Montgomery.	Muzaffargarh.	D. Ismail Khan.	D. Ghazi Khan.	Bannu.	Peshawar.	Kohat.	Hazara.
Latitude			32 0	30 6	32 58	34 20	33 37	34 10
Longitude			71 5	70 42	70 35	71 29	71 30	73 0
Height			600	..	...	1056		
No. of Years	6	6	6	6	6	6	6	6
January	·1	·5	·2	·4	·5	·8	·3	2·9
February	·2	·3	·6	·1	1·1	1·2	2·1	4·8
March	·7	·6	1·2	1·0	1·6	1·4	2·8	6·6
April	·2	·4	·5	·4	1·8	1·8	1·9	2·8
May	·1	·1	·4	·0	·5	·5	1·1	2·9
June	·6	2·0	1·2	·6	1·0	·2	1·1	4·8
July	3·7	1·0	1·4		1·4	1·1	4·0	9·8
August	2·5	1·7	·7	·0	2·1	2·1	2·3	6·9
September	1·9	·7	·5	·5	·3	1·5	1·6	3·6
October	·0	·0	·0	·0	·0	·3	·5	·9
November	·0	·0	·0	·0	·0	·0	·0	1
December	·3	5	·3		·2	·8	·5	1·7
Mean Year	10	8	7	6	10	12	19	48
Min. Year	4	3	5	2	6	8	9	32
Max. Year	27	14	11	2	16	17	29	69

VI.—Madras.—Mean Monthly Rainfall before 1861.

Place	C. Comorin.	Allepy.	Vaurius.	Trevandrum.	Palamcottah.	Quilon.	Cochin.	Shenkotta.	Trichinopoly.	Coimbatore.	Dodabetta.	Anjarakandy.	Cannanore.
Latitude	8 4	9 31		8 28	8 43	8 53	9 11	9 17	10 20	11 0	11 25	11 40	11 52
Longitude	77 4	76 20		7 2	77 48	76 39	76 0	78 10	77	7 1	77 25	75 40	75 30
Height	50	30	60	30	120	40	4		250		640	..	5
No. of Years	5	6	6	5	10	6	6	5	7	3	2	14	7
Actual Years	843–47	1841–46	841–46	1842–46	1842–46 -59	1841–46	1841–46	842–46	1842–48	844–46	1847–48	1810–23	842–48 1850–59
January	·2	2·4	·9	·8	2·7	·9	·7	1·3	2·0	·7	·1	·0	3
February	·0	1·2	·4	·2	1·2	·3	·0	·3	·2	·2	7·4	·1	·2
March	·9	4·3	·1	1·8	3·6	1·9	2·0	1·6		·8	3·6	·3	·4
April	·8	3·4	·4	3·5	10·7	3·5	3·2	2·2	1·4	·5	9·8	1·2	·7
May	4·6	26·1	4·0	10·2	5·8	16·5	17·8	3·6	2	·8	4·9	6·9	13·6
June	4·6	26·0	2·0	13·1	·7	17·9	30·7	4·3	1·3	·5	4·6	31·1	36·0
July	·7	18·2	·1	6·4	11·0	19·9	17·3	3·9	2·6	1·6	7·4	38·9	32·5
August	·4	9·2	·1	3·3		26·1	13·0	1·1	1·9	·2	9·3	22·5	16·1
September	·4	7·0	·9	3·5	1·6	3·3	4·8	1·8	5·3	5·3	7·5	12·3	·8
October	8·8	12·8	7·3	12·3	9·6	9·9	9·7	8·2	5·6	4·5	12·5	6·2	9·0
November	·9	6·0	2·6	4·1	14·7	3·9	2·3	5·9	2·6	·5	·9	2·5	2·2
December	2·8	2·9	2·9	4·5	5·8	1·7	3·5	3·0	4·5	·0	2·3	1·2	7
Year	28	119	25	65	57	86	105	37	3	2	01	123	121

VI.—Madras *continued*, and Maisur.

Place	Mercara.	St. Thomas Mount.	Punamalli.	Fort St. George.	Madras.	Nellor.	Haribar.	Karnul.	Masulipatam.	Gantur.	Ballari.	Samulkotta.	Vizagapatam.	Barhampur.
Latitude	12° 24′	13° 0′	13° 2′	13° 4′	13° 6′	14° 20′	14° 31′	15° 50′	16° 10′	16° 20′	17° 0′	17° 4′	17° 41′	19° 20′
Longitude	75° 48′	80° 15′	80° 10′	80° 14′	80° 21′	80° 0′	75° 51′	78° 5′	81° 13′	80° 30′	77° 10′	82° 14′	83° 21′	84° 50′
Height	4500	60	..	...	.	50	1831	800	..	100	1500	50	..	112
No. of Years	3	7	7	31	37	2	3	3	7	5	19	3	3	2
Actual Years.	1838–40	1842–48	1847–48 1855–59	1811–41	1804–7 1813–45	1859–60	1857–59	1857–59	1842–48	1855–59	1841–59	1857–59	1847–48	1859–60
January	·0	1·9	·1	1·2	1·1	·6	·0	·0	·2	·0	·2	·0	·2	·0
February	·4	·3	·1	·1	1·2	·0	·0	·0	1·6	·0	·0	·0	·3	·1
March	1·5	·4	·0	·7	·4	·0	·1	·0	·3	·9	1·0	1·0	·2	·0
April	2·6	·3	1·2	·4	·5	1·5	·6	2·7	·2	2·5	1·0	2·5	2·3	·0
May	7·4	2·5	1·9	1·1	2·4	·9	·4	8·1	1·9	4·6	2·1	1·0	2·0	·0
June	30·4	·9	2·0	1·5	1·6	1·5	·8	14·4	4·5	4·1	1·8	6·1	3·6	·0
July	55·9	2·3	4·3	3·7	3·1	8·7	·3	17·2	4·6	5·9	1·7	10·0	3·2	10·3
August	27·0	2·4	3·8	4·8	4·6	5·1	·9	8·4	4·9	7·7	3·0	6·5	4·0	7·1
September	11·9	3·7	4·5	4·3	5·1	3·4	·1	8·0	5·9	5·7	3·8	5·6	6·4	7·1
October	4·6	8·4	11·3	11·1	10·0	4·3	·5	8·7	6·7	7·0	5·6	9·7	14·2	2·4
November	1·4	10·2	14·3	14·1	12·4	32·1	·1	4·4	1·1	2·2	1·1	·0	4·0	·1
December	·2	8·4	6·1	4·6	5·1	2·0	·0	·0	1·9	1·0	·4	·5	·5	5·9
Mean Year	145	42	50	48	46	60	4	72	34	42	22	43	41	33
Min. Year	...				18									
Max. Year	...				89									

Maisur.

Place	Utacamand.	Kotagheri.	Dodabetta.	Mercara.	Bangalor.	Mysor.	Tomkur.	Shemuga.
Latitude	11° 25′	11° 20′	11° 30′	12° 25′	12° 58′	12° 18′	..	13° 57′
Longitude	76° 45′	77° 0′	77° 0′	75° 40′	77° 39′	76° 40′	..	75° 30′
Height	...	..	..	...	3030	...	...	...
No. of Years	7	1	11	5	30	30	30	30
Actual Years.								
January	·2	2·0	1·5	·0	·3	·1	·1	·0
February	·4	3·0	1·2	·0	·2	·2	·3	·2
March	1·0	6·0	2·2	·3	·6	·5	·6	·2
April	4·2	10·0	5·7	2·1	1·4	2·7	1·5	1·9
May	5·8	2·0	7·9	6·4	5·0	5·5	4·7	3·2
June	6·5	2·0	6·2	15·3	3·2	1·6	2·6	4·0
July	4·1	4·0	10·3	34·0	3·9	2·2	3·4	6·1
August	5·1	2·0	12·5	23·4	6·0	3·2	5·0	3·7
September	6·6	2·0	8·5	7·5	6·3	4·1	5·3	2·7
October	8·3	10·0	13·0	9·1	6·0	6·7	7·2	4·9
November	4·1	2·0	8·0	1·5	1·6	1·8	1·1	1·1
December	1·4	5·0	4·7	·1	·8	5	·3	·3
Mean Year	48	50	82	100	35	29	32	28
Min. Year	..	..	61	80	16	12	14	15
Max. Year	..	...	101	119	55	53	57	43

VII.—Minor Provinces.—Mean Monthly Rainfall of 24 places in the Central Provinces

Place	Saugor	Damoh	Mandla	[illegible]	Hoshangabad	[illegible]	Jubbulpore	Betul	[illegible]	[illegible]	Nagpur	[illegible]
Latitude	23 40	23 45		22 0	22 45	22 57	23 [illegible]	21 52	22 2[illegible]		22 3[illegible]	21 08
Longitude	78 48	79 25		79 3[illegible]	77 40	75 36	79 5[illegible]	77 5[illegible]	78 2[illegible]		82 30	76 13
Height ...	1944			1[illegible]	1019		1351		354[illegible]			
No. of Years	1	1	1	1	14	1	18	6	1	1	4	1
Actual Years	1871	1871	1871	1871	1855–68	1871	1851–67	1862–68	1871	1871	1865–68	1871
January	1·0	·6	4·5	3·2	[illegible]	1·0	·1	2·0	·5	·4	·1	·8
February	·4	·1	·3	·0	·6	2·4	·1	·5	·2	·4	·8	·0
March	·0	·0	·0	·0	·3	·0	·0	·0	·0	·0	·9	·0
April	·1	·2	[illegible]	·0	·1	·0	·3	·0	·0	·0	·6	·0
May	1·1	1·1	1·3	·6	·4	1·9	·0	·4	1·0	2·8	·0	·5
June	14·7	11·9	14·3	2·4	6·1	20·2	10·2	11·4	24·5	11·4	9·7	2·4
July	24·1	13·9	11·0	3·4	16·5	20·7	25·4	12·2	38·4	17·6	19·9	2·6
August	15·3	13·6	6·1	3·9	10·4	9·8	14·5	3·7	13·1	10·5	12·3	·6
September	11·4	14·8	13·2	7·6	6·1	13·2	14·7	15·6	27·7	8·2	[illegible]	
October ...	·0	·0	·0	·0	·7	·0	·2	·0	·0	·0	[illegible]	·0
November	·3	·0	·0	·2	·3	·0	·0	1·2	·0	·0	·6	
December	·5	·0	·0	·1	·1	·0	·0	·0	·8	·0	·6	·0
Year	69	56	50	21	48	70	56	47	1·6	51		
Previous Mean	55	48	51	[illegible]		[illegible]						

Place	[illegible]	[illegible]	[illegible]	[illegible]	Bhandara	Raipur	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Latitude	21 47	22 [illegible]	22 0		21 10	21 1[illegible]		19 50	18 40	22 0	21 2[illegible]	21 31
Longitude	78 18	78 5[illegible]	79 33		79 45	79 11		79 10	79 40	82 22	81 42	84 [illegible]
Height ...			2030			1025		632			98[illegible]	461
No. of Years	1	10	9	1	1	10	1	6	6	1	3	5
Actual Years	1871	1859–68	1868–69 / 1862–65	1871	1871	1859–68	1871	1863–68	1863–68	1871	1866–68	1864–68
January	1·2	1·4	1·0	·2	·0	1·0	·1	·0	1	·0	·7	·6
February	1·0	·3	·4	·7	·4	·2	·1	·0	·9	·1		
March	·0	1·0	1·2	·0	·0	·6	·0	·0	·9	[illegible]	[illegible]	[illegible]
April	·0	·7	[illegible]	·0	·2	·8	·1	·5	·4		[illegible]	
May	·8	·8	1·0	·9	·8	·9	1·3	1·0	1·8		[illegible]	
June	10·4	7·6	7·7	10·5	17·6	8·8	4·8	14·1	5·0	[illegible]	[illegible]	[illegible]
July	7·0	11·8	15·8	21·2	19·2	13·5	13·9	11·2	14·4			
August	3·1	10·9	12·0	13·9	3·8	9·6	1·2	11·1	[illegible]	[illegible]	[illegible]	[illegible]
September	3·3	[illegible]	[illegible]	[illegible]	[illegible]	6·6	[illegible]	[illegible]			[illegible]	[illegible]
October ...		·9				1·0						
November	[illegible]				[illegible]	·1					[illegible]	[illegible]
December	·0			[illegible]		·1	[illegible]	[illegible]	[illegible]			
Year												
Previous Mean		44	4[illegible]	67		41	[illegible]	4[illegible]			4[illegible]	4[illegible]

VII.—Minor Provinces *continued.*—Mean Monthly Rainfall in Haidarabad and Barar.

Place ...	Sicandarabad.	Sicandarabad.	Pusad.*	Wun.*	Basim Town.*	Mehkar.*	Janaphal.	Darwa.*	Chiki.*	Yotmal.	Karinja.*	Buldana.
Latitude	17 2	17 27	19 55	20 3	20 7	20 10	20 18	20 19	20 21	20 24	20 29	20 34
Longitude	78 28	78 23	77 39	79 1	77 12	76 38	76 39	77 50	76 19	78 12	77 3	76 24
Height ...	1830	1830	..	863	...	...	2050	1255	...	1583	...	2174
No. of Years	10	20	3	11	10	6	3	8	6	6	4	3
Actual Years	1861–70	1841–60	1868–70	1860–70	1861–70	1865–70	1865–67	1863–70	1865–70	1865–70	1860–63	1868–70
January	·3	·2	5	·3	·4	1·0	2·6	·2	·9	·6	·0	2·0
February	·2	·4	0	·3	·2	·2	0	·2	·0	·0	·0	·0
March ..	1·1	1·1	·4	·9	·3	1·1	1·2	1·0	2·3	·6	1·0	·3
April .	·5	·9	·0	·0	·0	·1	·5	·0	5	·0	·1	·0
May	1·3	2·2	·1	·4	·0	·2	·0	·1	·1	2·4	·6	4
June	3·8	3·4	3·3	7·1	6·0	4·8	3·9	6·8	6·0	9·5	3·9	8·2
July	5·6	4·9	10·9	10·9	10·3	8·9	9·3	6·9	6·0	15·6	9·4	8·5
August .	5·8	5·1	5·3	10·4	6·0	6·7	11·0	7·2	6·6	12·1	9·0	6·2
September ...	5·5	4·6	4·8	7·5	4·6	2·5	·9	7·3	3·2	4·9	4·5	4·2
October	2·6	3·9	2·3	1·2	·8	2·1	1·8	·9	2·2	·9	1·4	4·3
November ...	·7	·7	·3	·2	·0	·2	·1	·2	·1	·2	·8	·6
December	·1	·1	·6	·2	·4	·0	·1	·3	·6	·2	·0	·2
Year . .	27	27	29	39	29	28	31	31	28	4–	31	35
Year . .	21	13	19	26	18	21	18	7	10	34	23	25
Year .	33	43	37	64	47	34	37	51	36	70	43	39

Place ...	Tullagaon.*	Akola.	Murtizapur.*	Malkapur.*	Umraoti Town.*	Umraoti Station.	Daryapur.*	Ellichpur.*	Morsi.*	Melghat.*	Punad.*	Basim Station.
Latitude	20 41	20 42	20 44	20 54	20 55	20 55	20 56	21 16	21 20	21 42	...	20 7
Longitude	78 9	77 4	77 26	76 17	77 49	77 49	77 23	77 35	78 5	77 13	..	77 12
Height ...	1051	930	...	...	..	...	...	...	...	..	...	...
No. of Years	11	8	7	3	7	11	3	11	10	4	5	4
Actual Years	1860–70	1863–70	1864–70	1868–70	1863–69	1860–70	1868–70	1860–70	1860–69	1866–69	1863–67	1865–70
January	·7	·4	·4	·1	·7	·7	1·0	·6	·5	·5	·0	·6
February	·0	·2	·1	·0	1	·1	·0	·1	·2	·0	·5	·0
March ..	·3	·9	1·1	·1	1·9	·5	·2	·7	·6	·5	·3	·4
April .	·0	·0	·0	·0	·2	·1	·2	·3	·0	·3	·1	·2
May	·2	·2	·0	·0	·7	·6	·0	·4	·6	·0	·1	0
June	4·3	3·8	5·4	7·1	5·8	6·5	6·6	4·5	6·0	6·3	3·6	5·0
July	7·4	6·6	5·8	5·6	6·8	7·8	11·1	7·7	11·6	11·5	9·4	12·9
August .	9·4	6·5	10·3	5·7	7·8	8·2	8·7	6·4	6·9	17·2	8·1	8·4
September ...	4·5	2·3	2·2	5·7	3·0	4·8	5·5	4·2	5·2	7·3	4·4	8·0
October	·8	1·2	1·4	2·8	6	·9	3·2	1·3	·6	·8	1·2	4·1
November ...	·0	·3	·1	·1	·1	·2	·1	·4	·1	·0	·3	·3
December	1	·6	·1	·3	·3	·2	·1	·1	·0	·3	·2	·0
Year . .	28	23	27	27	28	31	37	27	32	45	28	40
Year . .	12	10	16	16	17	18	30	18	15	31	22	35
Year .	49	40	38	37	40	41	43	40	46	74	38	44

* From Native registers superintended by magistrates.

VII.—Minor Provinces *continued.*—Mean Monthly Rainfall in British Burma.

Place	Arakan.				Pegu.					Tenasserim.						
	Akyab.	Akyab.	Kyouk Phyu.	Sandoway.	Rangun.	Bassim.	Henzada.	Prom.	Thayatmyo	Maulmain.	Tavoy.	Tavoy.	Mergui.	Mergui.	Shway-gyeen.	Toungu.
Latitude	20° 8′	20° 8′	18° 10′	17° 10′	15° 20′	15° 30′	16° 4′	17° 13′	20° 18′	14° 8′	14° 7′	14° 7′	12° 27′	12° 27′	...	...
Longitude	92° 57′	92° 57′	90° 40′	91° 5′	92° 30′	90° 55′	92° 4′	92° 15′	92° 46′	93° 55′	98° 18′	98° 18′	98° 42′	98° 42′	...	...
Height	21	21	...	...	...	...	...	...	240	...	12	12	200	200	...	...
No. of Years	11	1	1	1	1	1	1	1	1	1	11	1	7	1	1	1
Actual Years.	Before 1869.	1872	1872	1872	1872	1872	1872	1872	1872	1872	1849–59		1853–59			
January	·4	·2	·0	·0	·0	·0	·0	·0	·0	·0	·3	·0	4·0	·5	·0	·0
February	·4	·0	·1	·1	·0	·0	·0	·0	·0	·0	1·8	·0	1·0	·0	·0	·0
March	·5	·0	·0	·0	·1	·0	·0	·0	·0	·0	1·2	·0	2·0	·8	2·1	·0
April	1·4	·1	·0	·0	1·0	1·1	·3	1·3	·1	·1	3·2	·6	3·0	3 3	1·7	·1
May	8·0	16·2	13·1	22·0	8·7	8·1	4·5	4·9	6·5	30·7	28·2	17·0	15·0	7·4	15·9	7·6
June	65·2	54·3	37·6	39·8	18·6	11·2	11·5	9·6	11·7	34·0	41·3	25·8	24·0	18·8	36·4	15·3
July	55·8	42·2	40·8	55·5	24·9	15·8	17·2	12·6	7·9	45·8	43·0	44·6	36·0	30·3	30·8	16·2
August	41·7	35·3	46·1	51·3	27·3	15·9	18·8	22·9	8·6	52·6	38·6	48·6	30·0	35·6	29·0	15·3
September	24·9	59·1	13·1	18·4	21·1	12·4	9·4	12·7	19·2	21·5	30·7	32·7	20 0	27·4	9·6	6·9
October	14·7	5·3	7·2	6·1	9·6	7·5	2·4	7·3	2·9	·8	10·2	7·9	24·0	10·5	11·3	9·2
November	6·3	7·0	1·8	1·0	1·2	1·7	1·4	0·5	1.2	·3	1·4	2·2	4·0	·8	1·0	1·3
December	·2	·2	·0	·0	·0	·0	·0	·0	·0	·0	·7	·0	2·0	·0	·0	·0
Year	219	220	161	195	114	74	66	72	58	186	201	180	165	136	138	72
1871	...	200	193	230	143	93	74	63	55	246	...	230	...	164	202	101
1870	...	177	157	..	79	87	...	46	42	185	...	185	...	...	184	65

DAY MAXIMUM RAINFALLS.

LONG CONTINUOUS FALLS,

AND SPECIAL RAINFALL DATA.

N.B. There are not any Day Maximum Returns for Bengal Proper. Day Maximum at Calcutta about 5 inches.

General data for extraordinary rainfall in Southern India, exclusive of very extraordinary cases.

	In 1 hour.	In 2 hours.	In 3 hours.	In 6 hours.	In 24 hours.
	Inches.				
For the sheltered table-lands of Balari and Kadapa	·75	·9	1·0	1·5	3·0
For the average of Plains like that of Tinnevelly, Ramnad, Trichinopoly, Eastern Coimbator, and Western Tanjor	1·0	1·2	1·4	2·0	4·0
For the bases of hills sheltered from the S.W., but more exposed to the N.E. mansun; and for a table-land like Maisur not shut out from the N.E. mansun	1·25	1·5	1·7	2·5	5·0
For plains and table-lands of Haidarabad and Nagpur	1·5	1·8	2·0	3·0	6·0
For plains like Tanjor, South Arcot, Chinglepat; for the plains of Gantur, Nellor, Rajahmandry, Ganjam, and Masulipatam	2·0	2·4	2·7	4·0	8·0
For the hills of Kadapa, Nellor, Gantur, Rajahmandry, Ganjam, and Masulipatam	3·0	3·6	4·1	6·0	12·0
For hill summits well exposed to the S.W. mansun	3·7	4·5	5·0	7·5	15·0

III.—Bombay Presidency.—Special Rainfall Data.

Day Maxima of Five Stations in Ten Years.

	1860.	1861.	1862.	1863.	1864.	1865.	1866.	1867.	1868.	1869.
Belgaum	20 October 2·54	17 July 3·29	22 October 2·59	26 June 4·75	19 July 3·80	22 August 6·07	17 June 4·85	18 July 2·28	6 June 3·01	28 June 5·51
Puna	11 June 3·12	2 July 3·10	23 October 3·11	25 October 2·27	10 June 2·94	4 June 2·75	3 October 2·06	16 October 7·90	8 June 5·00	28 July 3·15
Bombay....	13 June 7·31	18 August 6·08	21 June 6·16	24 July 8·24	26 June 3·56	2 July 8·04	3 August 8·08	16 July 5·69	25 August 10·51	27 June 15·31
Karachi .	14 May 0·93	24 August 2·30	31 July 1·48	9 July 5·27	21 July 2·57	29 January 2·22	5 August 4·83	16 August 1·02	30 July 1·10	21 July 6·74
Deesa .	.	24 August 8·19	27 July 2·56	13 August 3·54	25 July 4·61	9 July 3·77	11 August 2·80	24 August 2·25	12 August 5·15	21 July 5·31

Mahabaleshwar	13·06	inches in 24 hours on	2nd Sept., 1833.
Sattara .	4·40	„ „	April.
Uttray Mullay . .	15·10	„ „	24 Oct., 1845.
Bombay........	9·43	„ „	10 July, 1844.
„	4·22	„ in 1 hour on	11 June, 1847.
„	12·00	„ in 36 hours in	July, 1859.
„ (near ?). . .	14·00	„ in 24 hours on	9 Aug., 1868.
„	15·31	„ „	27 June, 1869.

IV.—North-West Provinces and Oudh.

Day Maxima in Six Years.

	1867.	1868.	1869.	1870.	1871.	1872.	Max.
Chakrata	...	...	3·1 June	2·5 Sept.	3·8 Aug.	N.B.—The Reports for 1872 do not give any Day Maxima.	3·8
Ranikhet	...	...	...	...	2·1 June		2·1
Dehra	...	4·3 July	4·9 Sept.	5·9 July	7·7 June		7·7
Rurkhi	4·8 Sept.	3·7 July	3·0 July	4·2 July	3·6 June		4·8
Mirat	...	..	2·5 Sept.	6·1 June	3·5 July		6·1
Bareli	...	..	8·8 Sept.	8·5 June	3·8 June		8·8
Agra	3·0 Aug.	0·7 June	2·8 Sept.	3·1 July	2·2 July		3·1
Fattahgurh	...	...	6·1 July	3·7 Aug.	3·0 July		6·1
Allahabad	...	.	.	3·9 Oct	7·8 July		7·8
Banaras	3·8 Sept.	4·6 June	2·1 July	3·7 July	3 4 June		4·6
Gorakpor	...	...	3·9 Oct.	4·8 July	5·7 Sept.		5·7
Naini Tal	8·0 Aug.	5·0 Aug.	7 9 Oct.	..	..		8·0
Lakhnau	...	4 8 July	7·5 July	5·6 Sept.	6·2 Sept.		7·5
Ajmir	3·0 July	2·2 July	4·2 Sept.	7 4 Aug.	3·9 June		7·4
Morar	..	..	2·9 July	...	..		2·9
Jhansi	4·4 July	3·5 Jan.	9·3 July	2·5 July	2·5 Aug.		9·3
Nagode	...	...	...	...	..		...
Naugong . ..	...	.	...	.	..		...
Bastee	...	...	..	6·4 July			6·4
Chunar	6·1 Sept.	...	...	...	...		6·1

Special Falls.

Lakhnau	9·00 inches in 48 hours on 9th and 10th July, 1869.				
Jhansi	15·00	"	36	"	23rd and 24th July, 1869.
"	3 80	"	24	" in	June, 1866.
Beawar (Ajmir) ...	5·70	"	"	"	January, 1864.
Rurkhi.	5·6	"	"	"	August, 1863.
"	4·8	"	"	"	September, 1865.
Naini Tal	8 0	"	"	"	December, 1863.
"	5 2	"	"	"	September, 1865.
"	5·4	"	"	"	July, 1866.
Ajmir	8·5	"	"	"	August, 1866.
Banaras...	2·8	"	"	"	September, 1863.
"	2·5	"	"	"	August, 1864.
"	6·6	"	"	"	July, 1865.
"	3·0	"	"	"	September, 1866.
Agra	2 8	"	"	"	July, 1863.
"	2·1	"	"	"	August, 1864.
" . .	3 8	"	"	"	August, 1865.
"	3 6	"	"	"	July, 1866.

V.—Panjab.

Day Maxima in Four Years.

	1869.	1870.	1871.	1872.	Max.
Delhi	3·4 Sept.	5·5 Aug.	3·4 Aug.	2·8 Aug.	5·5
Gurgaon	3·5 July	2·5 July	1·7 Sept.	2·7 June	3·5
Kurnal	3·2 July	1·5 Aug.	4·4 July	3·4 June	4·4
Hissar	1·6 Sept.	3·1 Aug.	1·8 June	4·5 Aug.	4·5
Rohtak	4·0 Sept.	2·8 July	2·6 Dec.	2·7 July	4·0
Sorsa	3·6 Sept.	1·8 June	1·5 June	2·7 Aug.	3·6
Ambala	2·7 July	5·0 Aug.	5·4 July	2·9 July	5·0
Ludiana	5·5 July	2·7 July	1·8 Feb.	4·8 Aug.	5·5
Simla	2·8 July	5·6 June	5·6 July	3·5 June	5·6
Jullundur	4·6 July	3·3 July	3·0 Feb.	4·4 Aug.	4·6
Hoshyarpur	2·4 July	3·4 Sept.	7·5 June	5·1 July	7·5
Kangra	5·2 Sept.	3·9 July	5·8 July	7·2 Aug.	7·2
Amritsar	4·4 June	3·9 Sept.	1·6 July	3·3 July	4·4
Sialkot	7·3 June	5·0 July	3·4 Aug.	2·5 July	7·3
Gurdaspor	4·6 July	4·0 Aug.	3·4 Sept.	2·1 July	4·6
Lahor	3·6 Oct.	2·1 Aug.	1·4 July	2·3 June	3·6
Firozpor	7·0 July	6·0 Aug.	2·4 June	6·5 Aug.	7·0
Gujranwala	2·6 July	2·8 July	2·8 June	3·9 Aug.	3·9
Rawalpindi	1·9 Sept.	3·2 July	1·9 Aug.	1·7 Jan.	3·2
Jhelam	1·8 Aug.	2·2 Aug.	1·5 July	1·2 July	2·2
Gujrat	3·5 July	2·9 July	2·4 July	3·2 July	3·5
Shahpur	1·4 Aug.	1·4 Aug.	2·8 June	3·2 July	3·2
Multan	2·9 Mar.	0·5 Aug.	0·9 July	3·5 July	3·5
Jhang	2·1 July	1·0 June	1·0 Dec.	1·8 Sept.	2·1
Montgomery	3·7 July	1·9 Aug.	4·0 July	1·4 July	4·0
Muzaffargarh	1·6 July	2·0 Aug.	1·6 July	2·3 Aug.	2·3
D. Ismail Khan	1·5 June	2·3 June	0·8 June	1·4 July	2·3
D. Ghazi Khan	2·7 July	1·5 June	1·0 July	1·5 Mar.	2·7
Baunu	1·5 July	1·0 Aug.	3·0 June	1·4 Apr.	3·0
Peshawar	4·4 Sept.	1·8 Aug.	2·9 Feb.	4·1 Aug.	4·4
Kohat	8·0 Mar.	5·0 Apr.	1·6 Aug.	2·0 July	8·0
Hazara	2·0 June	2·9 June	2·9 July	3·7 July	3·7

VI.—Madras, Maisur, and Curg.

Day Maxima at Madras between 1822 and 1857.

7·88 inches on 4th November, 1822.
8·87 „ „ 29th October, 1825.
12·08 „ „ 9th May, 1827.
7·77 „ „ 27th November, 1827.
7·50 „ „ 31st October, 1836.
9·65 „ „ 20th November, 1836.
7·20 „ „ 27th December, 1845.

17·00 in 12 hours on 21st October, 1846.
20·58 in 24 hours on 21st October, 1846.
11·45 in „ 4th May, 1851.
7·90 in „ 4th November, 1851.
6·22 in 5 hours on 20th November, 1856.
12·21 in 12 hours on 24th October, 1857.
18·04 in 24 hours on 24th October, 1857.

Falls at Bangalor.

4·85 inches in 24 hours in Sept., 1852.
1·00 „ in 35 min. in May, 1859.
2·71 „ in 24 hours in Sept., 1859.
2·27 „ in 24 hours in Aug., 1860.
1·20 „ in 15 min. in Sept., 1860.
1·60 „ in 40 min. in May, 1861.
2·49 „ in 2½ hours in Sept., 1861.
3·21 „ in 24 hours in Nov., 1861.

Longest Continuous at Bangalor.

10 days in July, 1859.
10 days in August, 1859.
10 days in August, 1860.
9 days in August, 1861.

Dodabetta ....4·30 inches in 24 hours in May, 1852.
Shemuga..................2·00 „ „ in April, 1859.
Shemuga..................4·00 „ „ in September, 1859.
Chittledrug ....10 days continuously in April, 1859.

VII.—Minor Provinces.—Haidarabad and Barar.

	1863.	1864.	1865.	1866.	1867.	1868.	1869.	1870.	Max.
Sikandarabad ...	1·70	3·00	1·90	2·65	2·05	2·74	2·27	2·00	3·00
Bassim Station ...	...	...	...	...	...	...	...	4·30	4·30
Bassim Town ...	...	...	...	...	...	..	...	7·20	7·20
Pusad	...	...	...	...	...	...	...	4·65	4·65
Janaphal	...	...	3·56	2·50	2·21	2 87	...	...	3·56
Yotmal............	...	...	6·30	4·30	6·40	4·35	3·60	7·40	7·40
Buldana	...	...	...	...	...	4·05	4·53	4·91	4·91
Akola	9·72	2·20	2·67	4·22	1·60	4·59	3·10	4·27	9·72

Longest Continuous Falls at Sikandarabad.

7 days 2·32 inches in 1863.
16 „ 5·50 „ 1864.
10 „ 2·11 „ 1865.
7 „ 1·17 „ 1866.

5 days 1·53 inches in 1867.
7 „ 3·23 „ 1868.
8 „ 3·38 „ 1869.

Central Provinces.

Day Maxima in 1869.

Nagpur	2·04	inches in	September.
Jabalpur	4·20	,,	July.
Raipur	9·70	inches in	July.
Seoni	4·10	,,	October.

Day Maxima in 1871.

Sagar	4·90	inches in	July.
Jabalpur	3·83	,,	August.
Hoshangabad	7·50	,,	September.
Pachmarhi	4·85	,,	September.
Seoni	2·50	,,	June.
Nagpur	4·02	inches in	June.
Chanda	4·00	,,	September.
Raipur	2·88	,,	September.
Sambalpur	2·88	,,	September.

British Burmah.

There are not any Day Maxima in the reports of this Province.

TABLES OF

HUMIDITY AND EVAPORATION.

I.—India.—Comparative Humidities for 27 places before 1861.

Place	Dodabetta.	Madras.	Belgaum.	Ballari.	Sicanderabad.	Satara.	Dapuli.	Mahableshwar.	Puna.	Serur.	Bombay	Dhulia.	Rajcot.	Ahmadnagar.
Latitude	11° 25′	13° 6′	15° 52′	17° 0′	17° 28′	17° 40′	17° 48′	17° 59′	18° 30′	18° 50′	18° 53′	20° 54′	22° 18′	23° 34′
Longitude	77° 5′	80° 21′	72° 42′	77° 0′	78° 32′	74° 2′	73° 16′	73° 30′	74° 0′	77° 25′	72° 52′	74° 45′	70° 50′	73° 1′
Height	8′40	27	2260	1500	1800	2320	600	4500	1800	1752	64	1000	450	1900
No. of Years	1	18	2	9	10	5	2	9	5	5	12	6	4	6
Actual Years	1847	1833–50	1858–59	1851–59	1850–59	1855–59	1858–59	1835–43	1856–60	1854–58	1847–58	1853 58	1857–60	1854–59
January	64	75	51	44	49	65	79	53	53	58	70	65	41	50
February	93	71	31	50	[illegible]7	62	79	44	40	42	67	52	35	43
March	70	72	44	34	45	56	80	40	44	15	67	45	33	38
April	81	76	48	60	37	48	80	41	43	25	72	46	35	35
May	87	69	63	51	38	60	85	59	57	43	68	47	40	43
June	93	64	79	58	49	79	58	83	67	50	80	65	55	63
July	93	68	84	71	60	75	85	88	79	75	85	76	64	75
August	93	72	84	60	72	89	90	94	84	71	80	85	72	71
September	93	76	79	45	80	84	89	83	79	71	85	85	60	79
October	93	76	62	67	63	74	80	88	63	53	76	71	45	63
November	93	79	62	79	55	66	89	64	52	38	67	59	33	58
December	86	75	54	65	51	69	70	60.	53	28	67	57	36	53
Average	87	73	63	57	53	69	80	66	59	48	74	63	46	56

I.—India.—Comparative Humidities for 27 places before 1861—*continued.*

Place	Karachi.	Deesa.	Haidarabad.	Thayat Myo.	Calcutta.	Fort William.	Hazaribagh.	Banares.	Kherwara.	Darjiling.	Mirat.	Landaur.	Peshawar.
Latitude	24° 51′	25° 14′	25° 30′	20° 18′	22° 34′	22° 34′	24° 0′	25° 17′	26° 42′	27° 2′	28° 59′	30° 27′	34° 20′
Longitude	77° 2′	72° 5′	69° 5′	92° 46′	88° 25′	88° 25′	85° 24′	83° 4′	79° 12′	88° 10′	77° 56′	78° 10′	71° 29′
Height	27	400	99	240	18	18	1900	270	1200	7000	900	7000	1056
No. of Years	5	3	2	1	2	6	3	2	5	3	1	4	3
Actual Years	1856–60	1857–59	1856–57	1859	1843–44	1854–59	1858–60	1858–59	1854–58	1857–59	1859		1858–60
January	49	52	59	41	70	68	49	52	67	77	77	49	67
February	57	46	50	32	63	66	53	56	57	85	73	65	69
March	53	36	44	27	69	67	51	41	50	80	69	45	77
April	54	34	43	...	70	68	50	36	37	80	40	42	74
May	65	45	44	58	77	72	76	56	35	87	53	53	48
June	65	56	50	68	81	80	52	66	49	94	59	58	53
July	72	72	59	80	85	85	55	95	64	94	77	78	56
August	76	80	65	76	90	85	80	85	90	94	64	88	61
September	68	72	68	76	90	85	75	85	72	94	80	78	49
October	57	51	60	80	80	80	62	90	67	75	84	65	57
November	39	37	48	70	80	70	50	66	68	74	78	57	58
December	47	44	56	66	74	64	73	72	63	84	62	54	67
Average	58	51	54	58	77	74	60	67	60	84	68	61	61

I.—INDIA.—Comparative Humidities (two observations) of 27 places between 1867 and 1873 inclusive.

Place	Galle.	Colombo.	Trincomalee.	Jaffna.	Port Blair.	Madras.	Hazaribagh.	Patna.	Goalpara.	Gorakpur.	Allahabad.	Jhansi.	Lakhnau.	Agra.
Latitude	6° 2′	6° 55′	8° 35′	9° 40′	11° 41′	13° 5′	24° 0′	25° 37′	26° 11′	26° 46′	25° 26′	25° 27′	26° 50′	27° 10′
Longitude	80° 20′	79° 45′	81° 25′	80° 0′	92° 42′	80° 70′	85° 24′	85° 8′	90° 40′	83° 18′	81° 45′	78° 37′	81° 0′	78° 5′
Height	40	42	175	9	61	27	2014	179	386	340	289	940	364	551
No. of Years	4	3-4	3-4	3	6	6	7	6	5	5-6	5-6	6	6	6
Actual Year.														
January	85	81	77	74	69	62	46	54	64	58	56	48	58	54
February	83	80	75	72	70	61	36	47	57	49	46	38	48	47
March	80	81	72	72	68	59	31	32	50	45	37	32	34	39
April	82	83	71	75	68	60	29	34	63	41	33	24	30	28
May	85	83	67	80	79	55	34	40	74	50	34	26	37	32
June	88	86	65	82	81	52	60	56	85	67	50	46	53	43
July	88	84	64	81	82	54	81	72	83	81	76	71	76	69
August	89	83	68	82	81	59	81	76	82	80	76	69	76	74
September	89	86	68	84	83	62	78	74	83	78	76	67	74	70
October	87	83	74	80	80	69	56	54	74	57	54	41	52	45
November	87	83	85	82	73	72	44	46	69	50	47	34	45	40
December	86	81	81	81	70	67	42	51	67	57	53	42	53	50
Average	86	83	72	79	75	61	51	53	71	59	53	45	53	49

I.—India.—Comparative Humidities (two observations) of 27 Places, between 1867 and 1873 inclusive—*continued.*

Place	Ajmir.	Bareli.	Mirat.	Sagar.	Jabalpur.	Hoshangabad.	Chanda.	Raipur.	Seoni.	Nagpur.	The following Three Places have true Mean Humidities given :—	Four Observations. Vizagapatam.	Four Observations. Banaras.	Four Observations. Rurkhi.
Latitude	26° 27′	28° 20′	29° 55′	23° 49′	23° 9′	22° 45′	19° 56′	21° 20′	22° 6′	21° 10′		17° 42′	25° 20′	29° 52′
Longitude	74° 40′	79° 25′	77° 40′	78° 48′	79° 59′	77° 46′	79° 19′	81° 42′	79° 33′	79° 11′		83° 22′	83° 2′	77° 56′
Height	1800	570	739	1944	1351	1009	632	980	2030	1025		31	263	878
No. of Years.	6	5–6	5	3–4	5	4	3–4	5	4–5	5		4	4–5	6
Actual Years.														
January	45	61	53	37	41	44	52	41	40	41		65	66	64
February	40	52	46	26	35	29	43	32	34	33		65	59	62
March	35	43	38	16	30	22	38	30	27	27		65	44	51
April	30	33	28	16	24	22	39	26	21	23		67	39	37
May	33	36	37	19	22	23	40	25	21	23		68	40	37
June	44	53	45	45	44	48	57	51	51	50		68	58	52
July	65	76	71	78	65	78	75	78	79	72		73	82	78
August	67	75	71	78	66	81	71	74	78	69		73	82	78
September	58	74	67	71	67	73	75	74	76	71		75	81	76
October	35	58	47	32	53	48	59	54	50	48		73	68	61
November	35	50	43	27	41	40	56	41	41	45		66	66	57
December	42	57	52	33	44	44	54	43	43	46		64	68	64
Average	44	56	50	40	44	46	55	47	47	46	Mean.	68	63	60

I.—India. Comparative Day Humidities (two observations) for various Stations of modern date.

	Bombay.					North-West Provinces.										
Place	Belgaum.	Puna.	Bombay.	Karachi.	Dera.	Chakrata.	Ranikhet.	Dehra.	Rurkhi.	Mirat.	Bareli.	Agra.	Fattehgarh.	Lakhnau.	Allahabad.	Banaras.
Latitude	15° 52′	18° 30′	18° 53′	24° 51′	25° 14′	30° 40′	29° 38′	30° 20′	29° 53′	29° 41′	28° 21′	27° 0′	27° 23′	26° 50′	25° 26′	25° 20′
Longitude	72° 42′	74° 0′	72° 52′	77° 2′	72° 6′	77° 55′	79° 29′	78° 8′	77° 54′	77° 41′	79° 27′	78° 0′	79° 48′	81° 0′	81° 52′	83° 0′
Height	2260	1800	64	27	400	7056	6143	2232	880	739	570	551	503	364	289	263
No. of Years.	10	10	10	10	10	4	2	5	5	4	5	5	5	5	4	5
Actual Years.	1860–69	1860–69	1860–69	1860–69	1860–69	1869–72	1871–72	1868–72	1868–72	1869–72	1868–72	1868–72	1868–72	1868–72	1868–72	1868–72
January	59	61	68	60	69	50	51	56	57	52	63	55	52	60	55	52
February . .	53	58	66	58	59	55	50	56	55	46	42	48	41	49	45	39
March .. .	46	50	67	61	50	48	39	48	41	38	45	39	32	35	37	29
April	48	47	70	63	47	40	39	41	29	29	36	29	31	28	35	26
May	59	52	67	66	53	43	51	36	29	37	36	27	29	35	34	30
June	80	73	78	62	64	66	71	59	49	49	56	44	50	55	57	53
July	88	80	85	74	75	88	82	79	69	70	73	67	76	75	75	73
August	88	82	86	76	83	91	84	81	69	71	72	72	74	76	75	72
September ...	84	78	84	72	67	84	77	76	65	66	74	68	71	75	76	73
October	74	71	78	51	63	58	48	55	46	46	56	44	44	53	55	50
November . .	62	61	76	53	58	45	43	47	41	42	51	38	39	45	46	45
December .	63	63	61	57	47	55	50	55	51	51	58	48	47	54	52	49
Average	67	64	74	64	62											

I.—India.—Comparative Humidities (two observations) of 27 Places, between 1867 and 1873 inclusive—*continued.*

Place	Ajmir.	Bareli.	Mirat.	Sagar.	Jabalpur.	Hoshangabad.	Chanda.	Raipor.	Seoni.	Nagpur.	The following Three Places have true Mean Humidities given :—	Four Observations. Vizagapatam.	Banaras.	Rurkhi.
Latitude	26° 27′	28° 20′	29° 55′	23° 49′	23° 9′	22° 45′	19° 56′	21° 20′	22° 6′	21° 10′		17° 42′	25° 20′	29° 52′
Longitude	74° 40′	79° 25′	77° 40′	78° 48′	79° 59′	77° 46′	79° 19′	81° 42′	79° 33′	79° 11′		83° 22′	83° 2′	77° 56′
Height	1800	570	739	1944	1351	1009	632	980	2030	1025		31	263	878
No. of Years.	6	5–6	5	3–4	5	4	3–4	5	4–5	5		4	4–5	6
Actual Years.														
January	45	61	53	37	41	44	52	41	40	41		65	66	64
February	40	52	46	26	35	29	43	32	34	33		65	59	62
March	35	43	38	16	30	22	38	30	27	27		65	44	51
April	30	33	28	16	24	22	39	26	21	23		67	39	37
May	33	36	37	19	22	23	40	25	21	23		68	40	37
June	44	53	45	45	44	48	57	51	51	50		68	58	52
July	65	76	71	78	65	78	75	78	79	72		73	82	78
August	67	75	71	78	66	81	71	74	78	69		73	82	78
September	58	74	67	71	67	73	75	74	76	71		75	81	76
October	35	58	47	32	53	48	59	54	50	48		73	68	61
November	35	50	43	27	41	40	56	41	41	45		66	66	57
December	42	57	52	33	44	44	54	43	43	46		64	68	64
Average	44	56	50	40	44	46	55	47	47	46	Mean.	68	63	60

I.—India. Comparative Day Humidities (two observations) for various Stations of modern date.

Place	Bombay.					North-West Provinces.										
	Belgaum.	Puna.	Bombay.	Karachi.	Deesa.	Chakrata.	Ranikhet.	Dehra.	Rurkhi.	Mirat.	Bareli.	Agra.	Fattehgarh.	Lakhnau.	Allahabad.	Banaras.
Latitude	15° 52′	18° 30′	18° 53′	24° 51′	25° 14′	30° 40′	29° 38′	30° 20′	29° 53′	29° 41′	28° 21′	27° 0′	27° 23′	26° 50′	25° 26′	25° 20′
Longitude	72° 42′	74° 0′	72° 52′	77° 2′	72° 5′	77° 55′	79° 29′	78° 8′	77° 54′	77° 41′	79° 27′	78° 0′	79° 48′	81° 0′	81° 52′	83° 0′
Height	2260	1800	64	27	400	7056	6143	2232	880	739	570	551	503	364	289	263
No. of Years.	10	10	10	10	10	4	2	5	5	4	5	5	5	5	4	5
Actual Years.	1860–69	1860–69	1860–69	1860–69	1860–69	1869–72	1871–72	1868–72	1868–72	1869–72	1868–72	1868–72	1868–72	1868–72	1868–72	1868–72
January	59	61	68	60	69	50	51	56	57	52	63	55	52	60	55	52
February	53	58	66	58	59	55	50	56	55	46	42	48	41	49	45	39
March ...	46	50	67	61	50	48	39	48	41	38	45	39	32	35	37	29
April	48	47	70	63	47	40	39	41	29	29	36	29	31	28	35	26
May	59	52	67	66	53	43	51	36	29	37	36	27	29	35	34	30
June	80	73	78	62	64	66	71	59	49	49	56	44	50	55	57	53
July	88	80	85	74	75	88	82	79	69	70	73	67	76	75	75	73
August	88	82	86	76	83	91	84	81	69	71	72	72	74	76	75	72
September ...	84	78	84	72	67	84	77	76	65	66	74	68	71	75	76	73
October .. .	74	71	78	51	63	58	48	55	46	46	56	44	44	53	55	50
November ...	62	61	76	53	58	45	43	47	41	42	51	38	39	45	46	45
December ...	63	63	61	57	47	55	50	55	51	51	58	48	47	54	52	49
Average	67	64	74	64	62											

I.—INDIA.—Comparative Day Humidities (two observations) for various Stations of modern date—*continued.*

	North-West Provinces.			Panjab.									Central Provinces.	
Place	Gorack-pur.	Jhansi	Ajmir.	Amritsar.	Dera I. Khan.	Dharmsala.	Lahor.	Ludhianah.	Marri.	Multan.	Rawalpindi	Sialkot.	Sagar.	Jabalpur.
Latitude	26° 46′	25° 27′	26° 27′	31° 40′	32° 0′	32° 8′	31° 34′	30° 55′	33° 35′	30° 12′	33° 4′	32° 29′	23° 49′	23° 9′
Longitude	83° 18′	78° 37′	74° 40′	74° 50′	71° 5′	76° 15′	74° 21′	75° 54′	73° 20′	71° 25′	73° 5′	74° 35′	78° 48′	79° 59′
Height	255	940	1800	...	600	...	1000	900	...	...	1700	900	1944	1351
No. of Years.	5	5	5	5	5	5	5	5	5	5	5	5	2	2
Actual Years.	1868-72	1868-72	1868-72	1867-71	1867-71	1867-71	1867-71	1867-71	1867-71	1867-71	1867-71	1867-71	1870-71	1870-71
January	58	42	44	61	51	77	56	54	50	48	50	50	44	45
February	50	37	39	66	50	66	53	54	60	45	50	45	26	40
March	42	29	35	48	47	45	48	47	45	36	44	46	19	28
April	41	23	30	...	38	49	51	32	51	28	36	40	19	23
May	52	20	23	...	30	54	23	30	39	21	32	24	26	26
June	68	41	45	58	34	17	32	39	60	26	29	35	54	53
July	80	66	63	53	45	87	48	55	74	34	41	57	78	67
August	81	66	66	60	49	87	51	54	68	38	46	62	76	67
September	80	64	57	50	45	70	53	55	60	35	43	54	70	71
October	59	39	37	35	39	68	32	34	43	32	33	37	38	56
November	51	30	35	37	45	...	36	32	43	33	40	41	31	45
December	59	38	43	54	50	...	48	44	64	41	46	50	34	39
Average	...	...	...	44	44	67	43	44	55	35	40	45	...	47

I.—INDIA.—Comparative Day Humidities (two observations) for various Stations of modern date—*continued.*

	Central Provinces.						Berar.			Maisur.	British Burma.	Madras.
Place	Hoshanga-bad.	Pachmarhi	Seoni.	Nagpur.	Chanda.	Raipur.	Umraoti.	Akola.	Buldana.	Bangalor.	The humidity returns for this province for 1872, are generally untrustworthy. Humidity of Akyab is included in the returns for Bengal.	There are not any humidity returns for Madras.
Latitude	° ′ 22 45	° ′ 22 27	° ′ 22 6	° ′ 21 10	° ′ 19 56	° ′ 21 20	° ′ 20 55	° ′ 20 42	° ′ 20 34	° ′ 12 58		
Longitude ...	° ′ 77 46	° ′ 78 27	° ′ 79 33	° ′ 79 19	° ′ 79 19	° ′ 81 42	° ′ 77 43	° ′ 77 4	° ′ 76 15	° ′ 77 39		
Height	1009	3540	2030	1025	632	980	1162	930	2174	3030		
No. of Years.	2	2	2	2	2	2	2	2	2	2		
Actual Years.	1870–71	1870–71	1870–71	1870–71	1870–71	1870–71	1871–72	1871–72	1871–72	1866–67		
January	52	51	44	46	59	46	38	53	53	53		
February	29	34	34	33	45	34	28	36	45	47		
March	24	9	33	29	52	30	18	28	32	46		
April	17	10	22	26	43	28	29	31	28	52		
May	26	30	22	24	40	25	40	34	38	56		
June	57	63	63	56	58	61	54	52	56	61		
July	80	86	80	73	78	79	68	61	74	65		
August	83	88	76	64	68	73	67	72	72	68		
September ...	72	81	75	67	75	76	68	75	76	69		
October	51	50	55	47	59	53	40	49	49	73		
November ...	46	50	46	50	59	43	31	49	25	61		
December ...	47	45	...	44	55	42	41	49	33	66		
Average	48	...	48	47	...	49	...	...	...	...		

N.B.—At this place the humidities were taken inside a house. [Bangalor]

II.—Bengal.—Mean Humidities (four observations) of 16 Places, from 1867 to 1873 inclusive.

Place	Akyab.	False Point.	Cuttak.	Sagar Island.	Chittagong.	Calcutta.	Jessor.	Dacca.	Silchar.	Hazaribagh.	Berhampur.	Gya.	Patna.	Monghir.	Darjiling.	Goalpara.
Latitude ...	20° 8′	20° 20′	20° 29′	21° 39′	22° 21′	22° 33′	23° 9′	23° 43′	24° 49′	24° 0′	24° 6′	24° 42′	25° 37′	25° 22′	27° 3′	26° 11′
Longitude ..	92° 57′	86° 47′	85° 54′	88° 5′	91° 50′	88° 21′	89° 7′	90° 27′	92° 50′	85° 24′	88° 17′	85° 2′	85° 8′	86° 30′	88° 18′	90° 40′
Height .	21	19	80	6	90	18	20	35	89	2014	65	347	179	160	6941	386
No. of Years Actual Years.	4-6	6	6	5-6	5-6	6	4-5	5-6	4-5	5-6	5-6	2-4	4-5	4-5	6	4-5
January ..	76	80	68	75	71	67	69	71	80	50	67	57	68	64	76	72
February ..	75	79	66	76	71	66	64	67	75	39	60	43	56	56	77	65
March .	75	83	65	78	74	67	63	70	74	36	54	27	46	44	68	61
April ..	76	86	66	80	80	72	68	77	77	33	62	33	44	47	75	73
May ...	81	85	67	81	82	75	77	84	82	40	67	47	50	54	81	79
June	87	84	75	84	86	83	82	89	88	65	80	61	63	72	90	88
July	89	85	82	86	88	87	87	89	88	86	86	81	81	84	91	86
August ..	87	85	83	87	88	88	87	89	89	85	85	81	83	84	92	86
September ..	85	83	83	86	87	87	86	88	88	83	84	79	80	84	90	87
October ...	85	79	78	83	83	79	82	81	86	64	76	65	70	71	77	81
November ..	81	72	69	75	78	71	76	73	82	52	71	52	62	62	72	77
December ..	78	75	68	75	74	69	74	71	81	51	69	57	64	62	71	75
Average true mean for year	81	81	72	80	80	76	76	79	82	57	72	57	64	65	80	77

Evaporation Data.

Bombay, 23 Years.

	Observed Mean Daily Evaporation.	Mean of 23 Years' Observations. Temperature of Air.	Temperature of Evaporation.	Relative Humidity.	Rainfall.	Mean Velocity of Wind in 1867.
	Inches.				Inches.	Miles per hour.
January	·240	74·0	67·5	·727	·05	10·8
February	·250	75·5	68·5	·708	·01	10·4
March	·283	79·2	72·4	·726	·00	12·1
April	·285	82·7	76·3	·754	·05	12·7
May	·322	85·3	78·6	·751	·56	10·0
June	·174	83·5	79·2	·842	20·84	15·7
July	·108	81·3	78·3	·877	24·26	19·4
August	·135	80·3	77·3	·873	13·16	18·3
September	·151	80·4	76·7	·858	9·64	12·7
October	·217	81·5	76·5	·810	2·20	9·8
November	·266	79·2	71·9	·715	·47	11·4
December	·270	76·1	69·1	·712	·05	9·0
Yearly Mean or Total	80·8	79·9	74·4	·780	71·30	12·7

Akola, 1870.

	Mean Daily Evaporation.	Total Monthly Evaporation.	Maximum Day Evaporation.	Humidities. Mean.	Humidities. Max.
	Inches.	Inches.	Inches.	Mean.	Max.
January	·258	7·75	·35	38	67
February	·359	10·77	·45	30	43
March	·472	14·16	·65	36	68
April	·741	22·24	·90	27	53
May	·945	28·34	1·11	32	64
June	·446	14·39	·96	42	62
July	·130	3·90	·36	60	77
August	·297	8·92	·42	56	75
September	·260	7·81	·46	70	94
October	·430	12·91	·95	62	95
November	·338	10·16	·47	58	97
December	·352	10·56	·48	51	71
Year Total or Max	·417	151·9	1·11	47	97

Various Evaporation Data.

Calcutta	Mean daily—	·50	inches.	
Rurkhi	"	·50	"	or 60 inches in 4 months.
Bangalor	"	·22	"	in November, vessel on ground.
Bangalor	"	·17	"	" vessel buried.
Vahar (Bombay)	"	·125	"	or 30 inches in 8 months (Conybeare).
India generally	"	·25	"	according to Col. Cotton, through the year.
Madras	"	·30	"	according to Observatory Data.

Evaporation Data for Pondicherry, by M. Lamairesse.

	Mean Evaporation, in Inches.			Mean Temperatures Corresponding.	Maximum Evaporation, in Inches.			Temperatures Corresponding.	Minimum Evaporation, in Inches.			Temperatures Corresponding.
	Day.	Night.	Both.		Day.	Night.	Both.		Day.	Night.	Both.	
1864.												
July	·255	·202	·458	88·0	·551	·394	·906	86·5	·118	·0 once	·157	86·0
August	·151	·144	·294	84·4	·236	·197	·433	84·0	·039	·0 twice	·039	81·9
September	·172	·142	·314	85·1	·276	·236	·433	85·3	·078	·0 once	·157	85·6
October	·135	·106	·240	83·3	·197	·197	·394	85·3	·039	·0 twice	·039	78·6 to 79·5
November	·137	·068	·205	80·2	·197	·157	·354	83·5	·0 once	·0 once	·039	74·3
December	·132	·071	·203	78·3	·197	·0	·276	76·5 to 79·7	·039	·0 twice	·039	77·9
1865.												
January	·169	·122	·291	77·7	·276	·197	·394	77·2 to 77·9	·039	·0 once	·039	76·3
February	·189	·125	·313	81·1	·276	·197	·394	81·7 to 83·3	·118	·078	·197	79·5
March	·188	·165	·353	83·3	·197	·197	·394	83·8	·157	·118	·276	83·1
April	·192	·157	·348	85·6	·276	·197	·394	86·2	·157	·039	·236	83·5
May	·305	·144	·449	86·0	·591	·197	·787	85·8	·157	·078	·276	84·7 to 85·8
June	·385	·127	·488	88·3	·551	·236	·591	88·7 to 89·6	·157	·0 once	·315	90·0
Year Totals	72·5	47·9	120·3		...	...	...					
Extremes	...	...	...		·591	·394	·906		0· once	·0 twice	·039	
Daily Mean	·199	·131	·329		...	...	...		...	...	...	

Evaporation and Corresponding Temperatures at Redhill, Madras. By Mr. Ludlow, C.E.

	Total Evaporation in Inches.		Daily Mean in inches.		—	Rainfall in Inches.
	In tank.	In open.	In tank.	In open.	Ratio.	
April	9·921	14·173	·409	·567	1·39	0·
May	11·181	14·252	·374	·476	1·27	2·559
June	11·772	15·079	·406	·500	1.24	5·591
July	10·079	12·008	·327	·386	1·19	7·047
August	7·205	8·465	·358	·421	1·17	1·890
Totals......	50·	64·	...	...	1·25	17·126
Means.....	...	...	·374	·469		

	Temperature of Water evaporated.		Temperature of Water in Tank.		Temperatures.	
	In tank.	In open.	At top.	At bottom.	In sun.	Of dew point.
April	83·7°	86·5°	89·1°	85·1°	106·8°	78·4°
May	81·2	82·8	84·2	81·1	102·0	78·8
June	79·3	81·7	82·0	78·8	101·3	77·1
July	79·9	81·9	83·0	80·6	97·3	77·0
August	80·4	82·0	81·7	80·2	98·4	76·6
Means......	81·0	83·0	82·3	83·0	101·1	77·5

Madras Observatory Evaporation Data.

	Mean daily.		Mean daily.
January	·300	July	·413
February..................	·305	August	·354
March	·359	September	·334
April	·392	October	·288
May	·460	November	·247
June	·484	December	·266

Total for year, 125·8 inches.

Evaporation at Chandarnagar, during 66 days of cold weather, from 27th September to 15th December, 1865, gave results varying between 24·8 and 35·4 inches; the observations being made on 60 tanks, whose surfaces varied from one quarter of an acre to 37 acres, and whose depths varied from 6 to 18·3 feet.

THE EVAPORATION DATA AND THEIR CONDITIONS OF OBSERVATION.

The data for Redhill, Madras, and for Pondicherry have been reduced to English measures from those given by M. Lamairesse, in Vol. XVIII., for 1869, of the "Annales des Ponts et Chaussées." With regard to the former, it is mentioned that the observations in the open were made at about 1000 yards from those in the tank, and that the results show that the decrease of the ratio of evaporation in the open to that in the tank is solely due to the diminution of the depth of water in the tank. The water in the tank went down in five months 75 inches, in spite of 8 inches of rain, in all 83 inches; of which the tank evaporator accounted for only 53 inches as lost by evaporation, hence only 30 inches were used in irrigation out of 83 inches in the tank—*i.e.*, three-eighths were utilized, and five-eighths lost.

M. Lamairesse also mentions that the English engineers in the Madras presidency also allow for a loss of water in irrigation by evaporation of 3 inches daily per square yard of land irrigated.

The conditions and mode of observation adopted in the old Bombay Observatory data, the Madras, Calcutta, and the Rurkhi data are not explained.

The Akola data were observed by a military surgeon; the evaporator being a simple tin pot, about four inches in diameter, surrounded by a little cotton-wool, and covered with a wire gauge covering to protect the water from animals; the water was measured every second day in the graduated measure used for measuring rainfall.

The conditions of observation adopted by Mr. Conybeare at Vahar are not forthcoming; but as his data more nearly represent the actual amount of evaporation from large sheets of standing water than those of others, and have been confirmed by practical results, they are exceedingly valuable.

ADDITIONAL

METEOROLOGICAL TABLES.

Averages of Monthly Mean Pressures deduced from the Registers of the years 1867 to 1873, for 28 Places in India, corrected to the Calcutta Standard.

Place	Galle.	Colombo.	Trincomali.	Port Blair.	Madras.	Vizagapatam.	Akyab.	False Point.	Cattak.	Sagar Island.	Calcutta.	Chittagong.	Jessor.	Dacca.
Elevation ...	40	42	175	61	27	31	21	18	80	6	18	90	20	35
No. of Years	3-5	4-5	3-5	4-6	6	4	6-7	7	6-7	7	7	7	6-7	5-7
January	29·838	29·851	29·744	29·847	29.972	29·949	29·987	30·028	29·965	30·014	30·012	29·911	29·993	29·977
February......	29·848	29·861	29·751	29·871	29·948	29·918	29 932	29·969	29·891	29·945	29·944	29·860	29·928	29·923
March	29·843	29·852	29·730	29·843	29·902	29·866	29·890	29·905	29·814	29·881	29·864	29·798	29·851	29·834
April	29·797	29·807	29·655	29·786	29·808	29·785	29·830	29·804	29·704	29·779	29·763	29·729	29·750	29·757
May	29·800	29·802	29·608	29·734	29·723	29·676	29·756	29·710	29·618	29·683	29·669	29·648	29·668	29·677
June	29·789	29·795	29·580	29 718	29·673	29·590	29·678	29·593	29·532	29·555	29·547	29·542	29·538	29·558
July	29·810	29·821	29·598	29·726	29·706	29·580	29·680	29·587	29·530	29·545	29·545	29·540	29·539	29·554
August	29·813	29·823	29·607	29·735	29·732	29·637	29·708	29·647	29·585	29·599	29·609	29·585	29·603	29·605
September ..	29·842	29·850	29·630	29·761	29·659	29·690	29·759	29·706	29·667	29·679	29·684	29·649	29·681	29·684
October	29·839	29·850	29·658	29·788	29·821	29·798	29·841	29·844	29·805	29·829	29·837	29·761	29·830	29·816
November ...	29·834	29·845	29·732	29·843	29·924	29·939	29·948	29·994	29 950	29·980	29·978	29·883	29·968	29·951
December ...	29·857	29·863	29·767	29·857	29 973	29 993	29·994	30·043	29·994	30·040	30·030	29·927	30·016	30·000
Year	29·826	29·835	29·672	29·792	29·828	29·785	29·834	29·819	29·755	29·794	29·790	29·736	29·780	29·778

Averages of Mean Monthly Pressures—*continued.*

Place	Silchar.	Hazaribagh.	Barhampur.	Patna.	Manghir.	Darjiling.	Goalpara.	Banaras.	Jhansi.	Lakhnau.	Rurkhi.	Agra.	Nagpur.	Jabalpur.
Elevation ...	89	1996	65	179	160	6912	386	263	836	364	880	532	1025	1353
No. of Years	4–5	6–7	6	6	5–6	5–7	5	6	6–7	6	7	7	5	5
January	29·930	27·986	29·948	29·848	29·857	23·384	29·606	29·786	29·165	29·682	29·116	29·499	28·942	28·616
February......	29·885	27·938	29·878	29·794	29·783	23·364	29·538	29·713	29·126	29·606	29·049	29·428	28·899	28·610
March	29·812	27·882	29·790	29·693	29·690	23·364	29·463	29·623	29·019	29·522	28·978	29·342	28·813	28·525
April	29·732	27·788	29·676	29·584	29·584	23·360	29·386	29·502	28·911	29·393	28·867	29·223	28·710	28·427
May	29·647	27·702	29·582	29·482	29·498	23·335	29·302	29·398	28·804	29·290	28·752	29·102	28·629	28·326
June	29·547	27·580	29·453	29·349	29·367	23·265	29·208	29·268	28·662	29·150	28·619	28·967	28·531	28·202
July	29·542	27·573	29·470	29·358	29·374	23·261	29·190	29·273	28·673	29·161	28·634	28·975	28·537	28·200
August	29·598	27·641	29·548	29·436	29·443	23·305	29·257	29·348	28·755	29·232	28·698	29·039	28·607	28·278
September ...	29·677	27·709	29·632	29·515	29·523	23·362	29·339	29·433	28·836	29·325	28·799	29·138	28·653	28·329
October	29·792	27·843	29·775	29·682	29·685	23·424	29·452	29·600	29·008	29·488	28·994	29·315	28·801	28·499
November ...	29·931	27·990	29·916	29·832	29·833	23·466	29·589	29·752	29·148	29·646	29·107	29·469	28·939	28·634
December ...	29·972	28·015	29·969	29·883	29·883	23·437	29·627	29·802	29·198	29·698	29·151	29·512	28·972	28·653
Year	29·755	27·806	29·720	29·621	29·627	23·361	29·413	29·541	28·942	29·433	28·894	29·251	28·753	28·442

Average Monthly Temperatures of 32 Places in India, from the Register of 1867–1873.

Place	Galle.	Colombo.	Trincomali.	Port Blair.	Madras.	Vizagapatam.	Akyab.	False Point.	Cattak.	Sagar Island.	Calcutta.	Chittagong.	Dacca.	Darjiling.	Monghyr.	Hazaribagh.
No. of Years.	4–5	4–5	4–5	6	6	4	4–6	6	6	5–6	6	6	5–6	6	6	5–6
January	78·2	80·0	78·7	78·9	77·0	76·8	69·9	71·3	70·9	69·3	68·8	67·5	66·8	42·5	62·7	61·6
February......	79·7	80·6	79·6	79·3	79·3	79·1	73·2	75·3	75·2	74·4	73·4	71·7	72·3	44·7	68·4	66·1
March	81·5	82·1	82·2	81·3	82·7	83·2	78·7	80·4	81·1	80·5	80·3	78·1	79·6	50·8	77·5	75·1
April	82·0	83·3	84·5	83·9	85·6	86·3	83·7	84·0	86·2	84·1	84·1	81·5	82·0	56·1	84·8	83·0
May	82·2	83·7	85·8	81·8	88·8	88·7	84·2	86·8	89·2	85·8	85·6	82·9	83·4	60·1	87·5	85·9
June	80·6	81·7	85·8	81·6	87·8	87·7	82·2	86·4	86·4	85·7	84·7	82·0	83·8	63·1	87·1	82·6
July	79·9	81·0	85·3	80·5	86·3	84·4	80·8	84·7	83·5	83·7	83·3	81·0	83·2	63·9	84·2	78·8
August	79·9	81·1	85·9	80·4	85·9	85·4	81·0	84·8	83·3	83·7	83·2	81·2	83·5	63·9	83·9	78·0
September ...	79·8	80·9	83·8	79·7	84·4	84·7	81·9	85·0	83·1	82·9	83·0	81·5	83·3	62·0	83·2	77·4
October	80·2	81·2	81·2	80·0	81·6	82·8	81·0	83·4	81·1	80·9	81·7	80·2	81·3	57·5	80·2	74·2
November ...	79·5	81·0	78·9	80·2	78·4	79·9	77·7	77·0	74·7	74·3	75·5	74·6	75·1	50·5	71·8	68·3
December ...	79·0	80·9	78·8	79·4	77·4	67·1	72·1	70·9	70·1	68·6	69·1	68·5	68·6	44·6	64·0	61·8
Year	80·2	81·5	82·5	80·6	82·9	82·9	78·9	80·8	80·4	79·5	79·4	77·6	78·6	55·0	77·9	74·4

Average Monthly Temperatures of 32 Places in India, from the Register of 1867–1873.—*Continued.*

Place	Patna.	Gorakpur.	Banaras.	Jhansi.	Allahabad.	Lakhnau.	Bareli.	Rurkhi.	Agra.	Ajmir.	Sagar.	Jabalpur.	Hoshangabad.	Nagpur.	Raipur.	Chanda.
No. of Years.	5–6	5	5–7	6–7	4–5	6	6–7	7	6–7	6–7	3–4	4–5	4–5	4·5	4–5	3–4
January	61·0	60·1	59·3	61·6	60·5	60·0	57·4	57·5	59·3	59·2	63·0	62·7	66·9	69·8	68·0	67·8
February......	65·9	65·3	67·3	69·3	65·9	66·4	62·8	62·1	65·5	66·5	68·8	66·5	71·4	74·7	70·6	73·9
March	77·2	75·1	76·4	77·8	77·8	76·0	73·2	70·6	75·7	75·1	78·4	74·6	79·1	81·8	79·7	81·1
April	84·3	83·6	86 5	87·2	85·4	85·9	82·0	81·1	85·8	85·8	84·3	84·4	88·3	87·5	87·7	89·5
May	88·1	88·7	91·5	94·1	91·2	91·7	88·4	88·1	92·5	91·8	88·1	90·6	93·2	93·4	92·5	91·7
June	88·2	87·2	91·2	91·6	90·0	91·7	89·2	89·9	93·1	91·7	84·6	88·1	88·1	86·8	86·7	86·8
July	84·0	82·9	85·7	83·3	83·9	86·0	83·9	84·4	86·1	84·8	76·8	78·6	78·9	78·6	78·2	79·2
August	83·8	82·3	85·0	82·0	82·3	85·9	83·1	83·9	83·8	82·4	75·4	78·1	78·7	78·9	79·4	79·3
September ..	83·0	82·1	84·0	82·0	82·2	83·5	82·0	82·5	82·9	84·3	76·1	78·6	79·5	78·7	79 4	78·6
October	79·0	77·8	78·8	78·8	77·8	78 6	76·1	75·5	78·9	79·4	75 9	75·8	77·2	76·8	77·0	75·9
November ...	69·8	68·8	68·7	72·1	68·7	68·4	66·3	64·3	69·7	70·9	71·3	66·8	72·0	71·7	70·7	71·3
December ...	62·4	61·0	59·9	63·7	61·3	60·6	58·8	57·8	62·1	63·6	65·4	62 2	68·7	67·9	67·4	67·5
Year	77·2	76·2	79·9	78·6	77·2	77·9	75·3	74·8	77·9	78·0	75·7	75·4	78·5	78.9	78·1	78·6

Monthly Temperatures of Solar Radiation for 16 Places in India, 1868 to 1873.

Place	Port Blair.	Jessor.	Hazaribagh.	Barhampur.	Patna.	Monghyr.	Rurkhi.	Lakhnau.	Allahabad.	Agra.	Ajmir.	Mirat.	Hoshangabad.	Chanda.	Akola.	Nagpur.
No. of Years	4-6	4-6	5-6	4-6	4-6	4-6	5-6	5-6	1	1	1	1	1	1	1	1
January	149.8	132.0	131.0	128.0	122.9	126.9	114.4	116.8	137	134	122	124	112	142	141	130
February......	149.3	136.9	136.1	136.4	131.6	135.3	121.5	127.2	145	137	132	130	119	151	149	136
March	156.6	144.5	143.0	141.1	143.0	145.8	133.2	139.0	155	153	145	142	127	159	159	147
April	157.4	147.0	154.8	150.5	150.4	153.4	143.6	151.4	164	165	155	153	142	166	166	155
May	146.8	148.5	158.9	152.6	154.0	155.7	149.3	156.0	163	161	157	152	144	167	170	158
June	137.4	141.2	151.5	141.6	150.2	151.8	148.0	151.6	161	164	158	157	139	161	156	150
July	138.9	142.2	144.1	142.0	143.7	147.7	138.8	141.2	143	149	141	145	118	153	143	128
August	142.8	143.7	146.3	145.8	142.9	146.3	137.1	143.0	147	142	147	143	121	139	142	137
September ...	144.0	145.0	147.4	144.4	142.4	146.3	136.5	138.5	149	142	146	135	113	139	140	129
October	148.2	144.7	141.3	142.8	140.7	142.9	136.1	135.8	155	151	151	136	124	151	155	141
November ...	148.4	138.0	136.5	136.3	134.1	136.0	127.5	129.1	148	142	137	130	116	152	150	136
December ...	149.2	132.7	129.6	127.3	125.3	128.0	117.3	120.4	138	135	127	120	111	146	...	131
Year	147.4	141.4	143.4	140.7	140.1	143.0	133.6	137.5								

Maximum, Minimum, and Mean Monthly Readings of exposed Grass Radiation Thermometer, at eight Places in India, in 1872.

Place	Akyab.			Cattak.			Chittagong			Manghir.			Hazaribagh.			Barhampur.			Darjiling			Rurkhi.		
	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.	Mean.	Max.	Min.
January	47	56	40	51	63	37	53	65	44	44	54	36	40	54	30	47	57	38	29	33	23	38	52	29
February	48	58	40	55	68	47	57	66	47	52	61	42	44	53	35	51	61	42	35	44	29	44	57	36
March	59	83	48	66	71	59	64	73	55	62	69	50	53	62	40	60	71	47	38	46	26	51	60	38
April	67	76	55	74	77	66	72	78	64	69	74	64	...	..	..	70	76	64	45	51	40	58	79	48
May	71	75	67	75	80	68	72	78	65	75	83	65	69	76	62	73	80	64	48	56	44	70	82	59
June	76	78	72	77	80	72	76	79	67	79	85	73	74	81	68	78	84	73	55	57	50	76	86	68
July	75	77	73	76	78	73	76	77	72	.	..	...	73	75	71	78	[illegible]	73	57	59	55	78	85	73
August	76	77	72	75	79	72	75	79	71	76	79	72	72	74	69	77	79	73	55	59	53	75	78	71
September	75	79	71	74	77	72	75	77	72	76	79	72	70	73	65	76	80	71	54	57	50	70	77	62
October	73	75	68	69	74	53	72	76	67	64	74	57	54	69	45	63	74	50	42	49	38	51	63	38
November	65	73	59	57	66	48	64	75	58	55	69	49	42	59	36	50	66	42	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Average Monthly values of the Tension of Aqueous Vapour, for 16 Places in India, between 1868 and 1873.

	Vizagapatam.	Akyab.	False Point.	Cattak.	Chittagong.	Calcutta.	Jessor.	Dacca.	Hazaribagh.	Berhampur.	Patna.	Manghir.	Darjiling.	Goalpara.	Banaras.	Rurkhi.
No. of Years	4	4-6	6	5-6	5-6	6	4-5	5-6	3-4	5	4-5	4-5	6	4-5	4	6
January	·608	·566	·621	·509	·494	·473	·423	·475	·297	·410	·351	·364	·206	·432	·338	·288
February......	·655	·615	·710	·560	·567	·545	·470	·527	·277	·428	·375	·391	·224	·453	·376	·331
March	·737	·743	·875	·670	·728	·691	·612	·697	·300	·502	·425	·403	·252	·501	·363	·349
April	·850	·879	1·015	·815	·868	·853	·772	·834	·406	·744	·533	·524	·334	·650	·441	·348
May	·920	·952	1·096	·906	·938	·934	·903	·949	·525	·843	·721	·671	·423	·785	·537	·444
June	·903	·961	1·076	·940	·966	1·000	·975	1·019	·747	·974	·889	·913	·525	·903	·807	·682
July	·863	·938	1·023	·953	·942	1·008	·985	1·013	·825	·991	·974	·977	·543	·926	·977	·893
August	·899	·943	1·028	·958	·952	1·010	·989	1·033	·815	·988	·972	·985	·545	·931	·961	·896
September ...	·893	·967	1·006	·949	·943	·992	·968	1·011	·797	·962	·933	·952	·506	·901	·919	·818
October	·825	·914	·919	·824	·871	·860	·875	·876	·578	·804	·741	·730	·365	·794	·624	·518
November ...	·676	·780	·678	·591	·676	·629	·609	·636	·359	·581	·473	·487	·264	·608	·449	·332
December ...	·586	·615	·584	·498	·524	·488	·454	·501	·292	·443	·369	·372	·210	·473	·358	·297
Year	·785	·823	·886	·764	·789	·790	·753	·798	·518	·722	·646	·647	·366	·696	·596	·516

Average Diurnal Movement of Wind in Miles at 16 Places in India—1867–73.

Place	Madras.	Vizagapatam.	Cattak.	Chittagong	Calcutta.	Jessor.	Dacca.	Silchar.	Hazaribagh.	Barhampur.	Patna.	Monghyr.	Banaras.	Rurkhi.	Nagpur.	Jabalpur.
No. of Years.	3	2	2-3	4-5	4-6	2-3	5	2-3	5	3-4	2-3	3	5	5-8	3-5	4-5
January	209.7	51.8	42.3	127.2	96.4	43.9	50.0	75.6	99.8	36.5	60.5	56.7	51.8	44.2	64.0	59.5
February......	181.7	68.1	60.9	133.4	104.3	51.4	64.1	96.3	125.8	42.3	72.8	61.3	70.7	55.0	80.9	55.7
March	206.2	98.2	86.3	155.1	135.7	75.6	106.0	103.2	149.2	61.7	96.2	74.2	74.2	56.1	86.0	71.9
April	261.5	124.9	125.9	184.0	201.5	118.5	150.0	92.7	164.8	98.6	144.2	99.9	70.4	62.5	117.1	84.5
May	286.0	125.7	129.0	174.5	204.7	118.8	155.4	84.4	190.4	114.7	138.7	91.7	91.5	79.2	147.7	113.2
June	292.3	120.7	122.3	192.2	199.4	150.3	211.3	89.5	206.2	144.3	90.3	97.2	96.9	84.0	167.0	144.8
July	273.4	117.5	95.9	190.5	150.0	116.1	197.9	91.3	178.1	112.5	81.2	79.0	93.6	66.6	172.2	130.2
August	229.0	74.2	73.0	155.8	129.6	97.4	148.6	91.7	148.5	85.6	84.0	76.7	76.7	49.4	126.5	110.7
September ...	218.4	73.3	67.4	129.4	127.7	73.4	124.7	85.8	141.4	81.4	95.9	55.7	66.9	34.7	118.2	90.4
October	170.9	64.5	52.8	92.3	86.1	51.1	58.9	69.9	102.8	44.3	54.7	30.7	51.1	23.3	85.9	50.9
November ...	222.1	82.6	33.8	100.6	83.1	44.6	38.9	73.4	84.3	31.9	52.4	31.7	24.7	22.2	71.8	47.1
December ...	224.9	60.4	35.8	112.8	91.0	49.9	44.2	59.5	84.5	26.1	44.0	36.3	38.5	27.2	62.3	52.5

Average Wind Resultants of 24 Stations in India—1863–73.

Place	Madras.		Vizagapatam.		Akyab.		False Point.		Cattak.		Chittagong.		Calcutta.		Jessor.		Dacca.		Silchar.		Hazaribagh.		Barhampur.	
No. of Years.	6		4		5-6		6		6		6		10		6		6		4-5		5-6		6	
	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.
January	73	N43E	47	S74E	38	N26W	30	N58E	16	N80E	50	N28W	40	N38W	57	N18W	40	N39W	35	S13E	56	N61W	55	N34W
February......	59	N88E	40	S22E	50	N21W	20	S29W	16	S10W	32	N38W	23	S79W	39	N53W	35	S79W	32	S24E	82	N60W	40	N64W
March	77	S50E	52	S45W	33	N47W	56	S50W	41	S17W	28	S69W	47	S33W	37	S80W	49	S24W	23	S39E	61	N78W	42	S81W
April	72	S40E	53	S39W	35	N83W	77	S42W	54	S10W	33	S17W	74	S3W	52	S11W	53	S8E	21	S64E	38	N76W	28	S1W
May	60	S14E	57	S42W	30	S57W	75	S34W	63	S1E	40	S14W	65	S12E	61	S12E	63	S23E	5	N84E	18	S85W	40	S41E
June	41	S54W	55	S50W	56	S1E	54	S47W	57	S25W	55	S26E	58	S5E	69	S16E	78	S20E	2	N79E	25	S27W	49	S31E
July	55	S59W	76	S74W	75	S9E	68	S64W	55	S41W	68	S43E	65	S12E	75	S15E	84	S22E	2	S13W	21	S17E	60	S43E
August	47	S57W	57	S69W	57	S1E	49	S64W	43	S45W	54	S36E	55	S18E	62	S14E	74	S12E	16	S33W	20	S43W	43	S45E
September ...	39	S41W	39	S33W	38	S9E	38	S39W	25	S	41	S38E	40	S30E	63	S21E	59	S14E	7	S22W	26	S52E	41	S56E
October	65	N39W	27	S78E	14	S39E	31	N49E	29	N35E	12	N22W	13	N53W	30	N59E	10	N66E	13	S17E	40	N57W	15	N9W
November ...	58	N20E	57	N74E	35	N14W	56	N25E	42	N14W	54	N14W	58	N19W	52	N6E	38	N13W	26	S75E	57	N46W	49	N28W
December ...	66	N25E	55	N77E	40	N15W	56	N32E	33	N12E	61	N23W	60	N27W	63	N9W	47	N31W	39	S53E	61	N56W	59	N26W

The Excess of the Observations in the direction of the Resultant is shown as a percentage.
The Direction of the Resultant is computed by Lambert's formula from the number of observed winds.

Wind Resultants—*Continued.*

Place	Patna.		Manghir.		Darjiling.		Goalpara.		Banaras.		Rurkhi.		Agra.		Jhansi.		Ajmir.		Nagpur.		Jabalpur.		Seoni.	
No. of Years.	5–6		4–5		6		5		9		9		6–7		3–6		6–7		5		5		4–5	
	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.	°/o	Dir.
January	45	N70W	52	S70W	20	S62W	35	S84E	30	N72W	15	N61W	27	N56W	20	N19W	10	N62E	34	N87E	29	N6W	21	N87E
February	41	N61W	39	S72W	24	S64W	35	S87E	37	N84W	23	N21W	36	N62W	20	N70W	14	S75W	18	N51E	17	N25W	12	N13W
March	53	N60W	29	W	25	S72W	36	N87E	40	N81W	24	N58W	37	N55W	12	S40W	21	S46W	12	S50E	18	S72W	6	N54E
April	36	N13W	23	N55W	26	S77W	44	S81E	40	N77W	11	N76W	29	N79W	28	S84W	52	S69W	23	N74W	19	N78W	23	N77W
May	38	N30E	40	N74E	21	S47W	48	S79E	20	N26W	8	S30W	26	N68W	33	N62W	73	S59W	44	N48W	37	N64W	30	N49W
June	35	N52E	63	N85E	21	S12W	27	S73E	8	N30W	9	S3E	21	N48W	18	N72W	67	S60W	52	N79W	63	S81W	41	S72W
July	36	N76E	57	S86E	27	S37E	31	S35E	8	S12E	33	S42E	20	N48E	33	S81W	69	S63W	70	S89W	67	S84W	67	S72W
August	19	S51E	23	S60E	24	S22E	18	S3W	7	S14W	23	S39E	14	S37E	17	W	53	S68W	59	N73W	66	S78W	44	S81W
September ...	34	S83E	38	S80E	15	S9E	26	S68E	3	S49E	4	S24W	17	N24W	38	N51W	84	S68W	41	N39W	36	N76W	33	N55W
October	9	N29W	14	S81W	18	S68W	46	S85E	32	N78W	8	S29E	27	N85W	28	N59W	48	S87W	66	N35E	10	N20E	59	N18E
November ...	25	N46W	46	N85W	14	S54W	67	S87E	34	N75W	4	S13E	21	S78W	25	N53W	11	N37W	65	N66E	24	N36E	55	N52E
December	41	N70W	54	S67W	21	S28W	55	S86E	21	N79W	12	N64W	20	N65W	20	N8W	8	N73E	57	N67E	22	N43E	37	N22E

The Excess of the Observations in the direction of the Resultant is shown as a percentage.
The Direction of the Resultant is computed by Lambert's formula from the number of observed winds.

Average Monthly Serenity of 15 Stations in India—1869-73.

Place	Vizagapatam.	Akyab.	Cattak.	Sagar Island.	Chittagong.	Jessor.	Dacca.	Silchar.	Goalpara.	Barhampur.	Hazaribagh.	Manghir.	Patna.	Banaras.	Rurkhi.
No. of Years	4	3-5	5	4-5	5	4-5	5	4-5	5	5	4-5	5	4-5	3-4	5
January	8·31	8·87	8·24	8·40	9·22	8·78	8·61	8·08	6·34	8·07	7·38	8·03	7·69	7·50	7·09
February	8·78	9·29	8·84	8·51	8·88	8·69	8·20	8·21	6·80	8·75	8·28	8·57	8·20	8·05	7·26
March	8·61	8·34	7·78	6·21	7·85	7·73	7·18	7·05	6·32	7·51	7·06	7·91	7·67	7·75	6·92
April	6·91	6·77	6·51	4·96	6·19	6·86	5·92	6·13	4·86	6·57	7·08	7·90	7·48	7·57	8·04
May	5·85	4·48	6·55	4·33	5·86	5·90	5·03	5·64	3·53	5·61	7·05	7·34	7·83	7·17	8·40
June	4·43	2·15	3·77	2·61	3·48	3·64	2·64	3·94	1·58	2·53	3·28	3·67	4·78	4·02	6·59
July	3·23	1·67	2·54	2·22	2·81	3·41	1·82	3·37	1·96	1·96	1·50	2·38	2·29	1·78	3·13
August	3·77	2·22	3·83	2·88	3·00	4·05	2·33	3·01	2·19	2·07	2·90	2·66	2·36	2·12	3·77
September	4·27	2·58	4·59	3·76	4·08	4·75	3·21	3·80	2·64	2·63	3·16	3·34	3·21	3·14	5·30
October	4·83	4·96	6·27	5·61	6·27	6·51	6·15	5·80	5·29	6·10	6·66	7·26	7·16	8·07	9·34
November	6·94	7·42	8·32	7·95	8·60	9·03	8·52	7·62	6·86	8·35	8·15	8·81	8·83	8·80	9·29
December	7·84	8·00	8·73	6·04	9·04	9·22	9·07	7·98	6·94	8·52	8·16	8·57	8·58	7·96	7·94
Year	6·15	5·56	6·33	5·29	6·27	6·55	5·72	5·89	4·61	5·72	5·89	6·37	6·34	6·16	6·92

In these tables an unclouded sky is represented by 10; an overcast sky by 0.
All the above are means of four observations daily.

GENERAL REMARKS

ON THE

METEOROLOGY OF INDIA.

I. RAINFALL.

The returns given, under the two heads of mean monthly rainfall, and day maxima, include everything available that would be of use to the hydraulic and irrigation engineer. The maximum and minimum annual rainfall for each place would have been given with the average annual rainfall in all cases, as in those of the Bombay Presidency, the Punjab, Mysore, and the minor provinces, but unfortunately they were not to be had. For the Madras Presidency, no mean monthly returns subsequent to 1861 are available. For Bengal and Burmah there are no day maxima procurable. In all cases hundredths of an inch of rainfall have been rejected, as in the first place they are unnecessary to the engineer, and in the second it would be aiming at a refinement of exactitude beyond the present powers of meteorological observation generally throughout India. As regards mean monthly returns, since it appears that the cycle of rainfall, in which maximum and minimum annual falls in India occur, is about ten or eleven years, the average for this number of years may be considered as practically correct; anything beyond that may therefore be considered unnecessary, and anything less as incomplete in that respect, and serving merely as a useful approximation.

The rainfall data given for years previous to 1861 were

extracted and reduced from a Parliamentary Blue Book published in 1863; those for years subsequent to 1861 were reduced from yearly returns furnished by the meteorological reporters of the various provinces of India; or rather from such of them as could be procured, including the latest supplied by the India Office in 1874. All returns made by natives under the superintendence of the civil officials, Anglo-Indian magistrates, and hence not under the control or inspection of meteorological reporters, or other qualified meteorologists, have been generally excluded from these statistics; those for Berar, reduced and examined by myself, forming the sole exception.

The position of the places mentioned, latitudes and longitudes, have, when given with elevations, been generally obtained from the yearly returns of the meteorological reporters; in other cases, that is, when the elevation is not given with them, they may be considered as mere approximations intended to guide the reader. As regards the nomenclature of the places, a serious difficulty in a country where there are no less than twelve main widely-spoken languages, it has been found impossible to adhere rigidly to one system; that of Sir William Jones, being strictly phonetic, and when once learnt, free from all doubt, is undoubtedly the best, and has hence been generally adopted; but as so many places, as, for instance, Bombay and Calcutta, have fallen into an English form, and might hardly be recognized in the Jones form of Mumbai and Kalkatta, the old established manner of spelling these and a few other names has been adhered to.

As to the grouping of the rainfall stations, it would no doubt have been far more correct meteorologically to collect them into natural groups, as shown in the table on the following page; but for many reasons this has not been considered advisable at the present stage of Indian meteorology, and hence the following territorial arrangement has been generally adhered to:—

I. India generally, irrespective of Province.
II. Bengal (under the Government of Bengal).
III. Bombay and Sindh.
IV. The North-west Provinces.
V. The Punjab.
VI. Madras (under the Governor of Madras).
VII. Minor Provinces—including the Central Provinces, Berar and Haidarabad, Oudh, Mysore and Kurg, British Burmah, and Ceylon.

In some cases, however, owing to the arrangements of the meteorological reporters, some of the data for places in the Minor Provinces will be found mixed up with those of the nearest large province, as Oudh with the North-west Province, Maisur with Madras, and Burmah and Ceylon with Bengal.

TABLE OF NATURAL GROUPS OF RAINFALL STATIONS.

No.	Group.		Position.		Rainfall	
			Latitudes.	Longitudes.	Mean Annual.	Maximum.
I.	Insular	Ceylon .	° 0 to 9	° 79 to 82	80	
II.	Detached	Burmese .. .	0 to 21	90 to 100	128	185
III.	The East Coast, or	Madras . ..	8 to 20	78 to 85	40	80
IV.	The West Coast, or . .	Bombay ..	8 to 24	66 to 76	80	312
V.	The Southern, or . .	Bangalor ..	8 to 15	77 to 78	50	120
VI.	. . ..	Belgaum ..	16 to 17¾	74 to 77	30	51
VII.	The South Central, or	Karnûl	16 to 18¾	77 to 78½	26	34
VIII.	. ...	Puna	18 to 19½	73¾ to 74½	24	72
IX.	. . ..	Nassik	19½ to 21	73¾ to 75½	52	83
X.	The Central, or . .	Nagpur	21 to 23¼	78 to 80	40	70
XI.	The North Central, or	Mount Abu	23½ to 25½	69 to 75	24	65
XII	The North Eastern, or	Cherrapunji.	20 to 27½	91¾ to 93	78	612
XIII.	. . .	Dacca	23½ to 26½	89½ to 91¼	93	
XIV	The Bengal Coast, or	Calcutta ..	22½ to 23½	88½ to 89½	93	
XV.	.	Danapur .	24¾ to 26½	85 to 85½	30	230
XVI	The Bengal Hill, or	Bhagalpur .	23¼ to 23¾	87 to 88¾	45	
XVII.	. . . .	Bardwan ..	23¼ to 23¾	87 to 88	30	
XVIII.	.. . .	Mursnidabad	23½ to 24¼	85½ to 88½	46	124
XIX.	....	Banaras .	25 to 26¼	81¾ to 83½	41	62
XX.	The Northern, or	Delhi ...	26 to 30½	76¾ to 79¾	30	96
XXI	The North Western, or	Lahor	30½ to 31	74 to 78½	17	57
XXII.	The Western, or	Indus .	25 to 30	71 to 73	9	30

As to the laws of the irregularities of rainfall over the vast continent of India, and their causes, nothing has yet been positively determined. The phenomena of the mansuns, and their causes, as well as those of the existence of the large comparatively rainless regions west of the Indus, have been familiar to every one for many years; famines, due to the periodic rainfall, being either in excess or in deficiency on the whole, or at the usual period of high rainfall, the rains being too late or too early, have existed for ages, and have continually decimated the population locally, without the causes being discovered. Sometimes the summer rainfall is thrown to the east, sometimes to the west of the Bay of Bengal:—sometimes it is scanty in Lower Bengal and abundant in Northern India, and sometimes the converse. After a few years, when a uniform and trustworthy system of meteorological observation shall have been extended all over India, it is very probable that these phenomena will be better understood: at present the record of pressure, temperature, and wind, &c., of the Presidencies of Bombay and Madras are practically inaccessible, and those of Northern India being irregular and untrustworthy, the only records that are of any value for this purpose are those under the control of Mr. Blanford, for many years Meteorological Reporter to the Government of Bengal.

From these he has been enabled to discover a most important law, viz., that the position of the circle of minimum barometric pressure in Bengal in March and April does, in connection with other meteorological data, furnish means for indicating roughly the amount and the distribution of the mansun rainfall of the year, which commences in May or June. We may, therefore, hope that in a few years it will be customary to announce every spring the probable amount and distribution of the summer rainfall over India, and thus save the large and continual losses of crops now due to a want of this knowledge.

Another most important law of rainfall, discovered by Mr. Meldrum, of the Mauritius, will probably be found to admit

of application to India. Mr. Meldrum, of the Mauritius, originally established the law that the years of minimum and maximum sun-spot frequency were coincident with those of cyclone frequency in the Indian Ocean, and has lately estabished the law of the coincidence of these years with those of minimum and maximum rainfall at Port Louis. Now the years of minimum sun-spot frequency are—

1833 1844 1856 1867,

and those of maximum sun-spot frequency are—

1837 1848 1860 1871,

denoting a cycle of between ten and eleven years. The rainfall at Adelaide, Brisbane, and the Cape of Good Hope, shows a similar periodicity of eleven years generally, but the epochs are not coincident with those of sun-spot frequency; these periodicities are, therefore, assumed to be the natural consequences of the same law.

Mr. J. Norman Lockyer, Superintendent of the Department of Science in Oudh, has attempted to apply these principles to rainfall in India; he states that the rainfall at Lakhnau was 64·6 inches in 1870 and 65·0 inches in 1871, each of these amounts being more than 22 inches above the fall of the preceding year 1869, or the two following years, 1872 and 1873, in which the falls were 41 and 34 inches; he also points out that the Madras rainfall records support the same law; they are thus:—

	Year	Inches.	Totals.
Minimum	1843	41·	125·
	1844	45·	
	1845	39·	
Maximum	1847	81·	175·
	1848	40·	
	1849	54·	

and show an interval indicative of a periodicity coincident with that of the sun-spots.

While, therefore, it will probably be long before meteoro-

logical science and spectrum analysis together combine to discover the nature of the connection shown by these facts, in the meantime the knowledge of the periodicity of the rainfall cycle may, like that of Mr. Blanford's theory previously mentioned, become an invaluable blessing to India.

At present, neither of these theories can be considered as established, indeed the periodicity of a cycle of sun-spot frequency is not yet fully proved. All that is yet established, as proving the connection between the solar-spots and the meteorological conditions of the earth, is, that the years of sun-spot frequency generally correspond to those of maximum solar radiation temperature, of the black-bulb thermometer in vacuo; while of the fact that variation of rainfall is caused by that of temperature there is no doubt. A widely extended series of meteorological observations, in all parts of the world, will be required before this connection can be made to yield useful results.

II. EVAPORATION AND HUMIDITY.

Next to the amount and distribution of rainfall, evaporation is among meteorological data the most important to the hydraulic engineer. It is not sufficient for him to know how much rainfall may be expected at any time and in any length of time, he wishes to know how much of this has to be provided against, or how much of it he can utilise, after all losses by evaporation and absorption are allowed for. These losses, then, require to be determined, not with any theoretical degree of exactitude, but with a practical degree of accuracy that will be sufficient security against gross error or gross waste. The large number of bridges in India that have been swept away for want of sufficient waterway, and the large amount of water valuable for irrigation that has annually been allowed to evaporate in shallow tanks, are painful examples of semi-barbarous engineering management, and ignorance of physical and meteorological conditions.

The evaporation data given in the tables are exceedingly few in number, and have mostly been conducted on false principles; they do not by any means truly represent local evaporation as regards absolute amount, but are relatively useful, yielding comparative results, which, in combination with a few absolute data, and a knowledge of comparative local meteorological conditions, may be made to yield roughly approximate absolute data for a large number of places. It is with this object that all available comparative humidity data for India have been given in the accompanying tables.

For example:—We will suppose that we require a rough approximation to the evaporation at Akola, a place for which comparative humidities are given. The most trustworthy data of absolute evaporation, representing evaporation from a large standing sheet of water, are those of Mr. Conybeare, at Vahar, near Bombay; they give thirty inches in eight months of hot weather, or about forty inches in a year. Now, the Bombay Observatory data give mean daily evaporation data, which are among themselves and under their own conditions relatively correct, although their sum total, eighty inches, is not true in representing absolute evaporation from a sheet of standing water. We can therefore tabulate proportional mean daily evaporation for Bombay that will be absolutely correct, thus—

Bombay.	Comparative Evaporation Mean daily.	Absolute Evaporation Mean daily.	Relative Humidity corresponding.
January	·240	·120	73
February	·250	·125	71
March	·283	·142	73
April	·285	·143	75
May	·322	·161	75
June	·174	·087	84
July	·108	·054	88
August	·135	·068	87
September	·151	·076	86
October	·217	·109	81
November	·266	·133	72
December	·270	·135	71
Mean for Year	·225	·112	78

And assuming that the observed humidities taken at Akola and at Bombay are daily means, *i.e.* of two observations in the twenty-four hours, at 10 A.M. and 4 P.M., they admit of comparison, and we can then tabulate the true evaporation for Akola, thus—

AKOLA.	Comparative Evaporation.	Humidity.	True Evaporation.	
January	·258	38	·129	
February	·359	30	·179	
March	·472	36	·236	
April	·741	27	·370	
May	·945	32	·473	Akola. Deduced total evaporation for the year, 75·7 inches.
June	·446	42	·223	
July	·130	60	·065	
August	·297	56	·149	
September	·260	70	·130	
October	·430	62	·215	
November	·338	58	·169	
December	·352	51	·176	
Mean for Year	·419	47	·210	

This assumes that the evaporation at Bombay and at Akola would be the same for the same relative humidity, viz , ·130 for 70, and the rest are therefore tabulated in proportion to the comparative daily evaporation data for each month, getting a total annual evaporation of 75·7 inches. This result, though confessedly an approximation, is sufficiently true to be useful to the hydraulic engineer, and is infinitely better than the old practice of basing comparisons of evaporation upon corresponding mean temperatures, or the still worse method of assuming that evaporation all over India is about the same. In this way also we adopt a means of utilising the various evaporation data, taken under such different conditions, that have been generally hitherto thrown aside as useless.

In the future, we shall probably have a widely extended series of evaporation observations taken all over India, under the orders of Mr. Blanford, now appointed to the new post of Meteorological Reporter for India. If these are conducted in a perfectly uniform manner, whether the evaporators are simple tinpots, double boxes, or masonry cisterns, we shall

possess most useful data for purposes of comparison, if the relative humidities and the average wind-movements be simultaneously observed ;—and from these, and with the aid of a few carefully conducted series of observations giving absolutely true evaporation, as from a sheet of standing water, we shall be able to tabulate absolute evaporation from any place in India with sufficient accuracy to serve the ordinary purposes of the engineer.

Before anything more than this can be expected, a large series of carefully conducted experiments must be made in order to ascertain more exactly than is known at present, both the relation between the amount of evaporation and the depth of the evaporating vessel or reservoir, and that between it and the velocity of the wind, in various stages of relative humidity ;—we shall then have results that will enlighten us considerably as to the conditions under which we may lose as much as half the water we store for irrigation in India.

The tables for humidity are intended to aid the engineer in determining evaporation data in the fore-mentioned manner; they may also be useful to the agriculturalist who requires certain hygrometrical conditions to suit various crops in different localities.

It is unfortunate that in many meteorological stations only two observations of humidity, viz., at 10 A.M. and 4 P.M., have been taken daily; their mean represents, therefore, only the mean of the day, exclusive of the night, and is not a true daily mean for the twenty-four hours. Such means are therefore only comparative means; the true mean is that of observations taken at equal intervals through the twenty-four hours. Those of observations taken six hours apart yield a mean differing only two per cent. from the mean deduced from hourly observations; those of observations taken at eight hours' intervals are far less correct. There are no means of deducing a true daily mean from the two observation humidities; these, therefore, only admit of comparison among

themselves. In some cases the relative humidities are recorded as percentages of saturation, in others as decimal fractions; it has been thought best to leave them in the form in which they were recorded, as this presents no difficulty.

A table showing the average monthly values of the tension of aqueous vapour for sixteen stations in India is given among the additional meteorological tables, which are placed apart from those that are more useful to the engineer, viz., those of rainfall, evaporation, and humidity.

The hygrometrical data are simply inferential results derived from observations with dry and wet bulb thermometers, no direct determinations of the dew point by Daniell's or Regnault's hygrometer having been practised. The calculations have been made by Guyot's tables, which are computed by August's formula with Regnault's constants. In Berar, Apjohn's formula was used, and the results were hence less accurate.

In explanation of the various hygrometrical conditions that are thus reduced to figures and statistics, we may, for the sake of those that wish to add their observations to the common stock in a useful form, offer a few remarks.

The wet and dry bulb thermometers used for observation are suspended in the open air, in a thermometer shed, screened from the wind, but exposed freely to the air, the object being to ascertain the ordinary humidity in still, unconfined air. The dry bulb thermometer shows the actual temperature of the air; the wet bulb being cooled by evaporation falls in temperature, and the difference of the readings of the dry and wet bulb increases with the rate of evaporation, and this again increases with the dryness of the air, although not in the same ratio. The wet bulb is never cooled to the temperature of the dew point, but both that temperature and the weight of vapour in the atmosphere, and the relative humidity, are obtained by calculation. The readings recorded are simply those

of the wet bulb and the difference between those of the wet and dry bulbs, except at hill stations in India, where a barometric reading is necessary in order to apply a correction. From these readings, taken at six hour intervals, and with the aid of Guyot's tables, useful mean humidities may be obtained.

The four most important hygrometrical elements are :—

I. The temperature of the dew point.

II. The actual amount of water mixed with a certain mass of air in the form of vapour.

III. The amount of water necessary to saturate a certain mass of air.

IV. The relative degree of humidity of the air.

The temperature of the dew point is that degree to which the temperature must be reduced in order to effect complete saturation of the air; if then the temperature of the air be higher than that of the dew point the air is not saturated, and if after complete saturation the temperature of the air declines rain must fall. The amount of water necessary to effect saturation varies with the temperature: at 32° air is saturated by a little more than two grains per cubic foot; at 42° by 3; at 49° by 4; at 56° by 5; at 61° by 6; at 66° by 7; at 70° by 8; at 100° by 20 grains nearly. The difference between the actual amount of water in the air, and the amount that it could hold at that temperature is the amount short of saturation; and the ratio between the same quantities is the relative humidity. For example :—At the temperature of 32° if there be one grain of water in a cubic foot of air the relative humidity is 50; at 100° there must be ten grains present, to give the same relative humidity of 50.

The formulæ used for obtaining these data from the readings of the wet bulb thermometer and the difference of the wet and dry bulb, are those of Dr. Apjohn and of August—the latter are more recent and more accurate; but to make use of them it also is necessary to have tables of elastic force of vapour corresponding to various temperatures. August's

formulæ, as given in Guyot's Tables, Smithsonian Collection, 1862, are for temperatures above freezing (1) and below freezing (2) respectively,

$$(1) \quad F = f - \frac{\cdot 48\ (t - t')}{1130 - t'} \times b.$$

$$(2) \quad F = f - \frac{\cdot 48\ (t - t')}{1240\cdot 2\ t' - t} \times b.$$

Where F is the elastic force of vapour at the dew point ;
f is that of saturated vapour at the temperature t' ;
t is the observed temperature of the dry bulb;
t' is that of the wet bulb ;
and b is the mean barometric pressure which is assumed $= 29\cdot 7$ for the plains of India generally by Mr. Blanford.

Having thus obtained F, the corresponding temperature at the dew point can be got from a table (Drew's Meteorology or Guyot's tables) based on experiments on vapour elasticities. To calculate the humidity, obtain from a similar table the elastic force of saturated vapour (F′) due to the temperature (t), then the humidity =

$$\frac{100\ F}{F'}$$

If, however, the humidity alone be required, it can be obtained direct from Guyot's humidity tables, as before mentioned, without any calculation.

From Indian hygrometrical data, it appears that the air is least moist upon the average of the whole year at about two P.M., but this varies at different seasons ; the greatest moisture in the day is at about six A.M., and there is a mean state about nine or ten o'clock, both A.M. and P.M. The extremes of humidity are generally the reverse of those of temperature as regards time, except in June and July, when the moisture is greatest about midnight; in August and September the increase of moisture after midnight is very small. The contrasts between the humidities of Madras and Bombay show the effects of the north-east and south-west

mansuns. The variations of humidity from year to year at the same place seem not to follow any law, and the humidities for various places seem not to be affected by latitude or longitude. The effect of elevation is everywhere clearly shown by the almost proportional lower reading of the dew point, less water being present in the air, a nearer approach to saturation, and a higher degree of humidity; but beyond this nothing can be inferred at present, and before any further deduction can be made, a series of *direct* determinations of the dew point at various high elevations will be necessary.

The places whose hygrometrical state seems to be nearest to that of England are Dodabetta and Darjiling. Landaur at the same elevation as Darjiling, has the same annual temperature of the dew point, and the amount of water present in the air is nearly the same; but as the amount of water necessary for saturation is three times as large as in England, the air is less humid. At all other places the dew point is a great deal higher than in England, and the amount of water actually present as well as that necessary for saturation is greater, so that the air is throughout the whole year, and more especially in the cold weather, much less humid. At certain places, Belgaum, Sattara, Mahableshwar, Dapuli, Bombay, Thayatmyo, Calcutta, and the country thence to Banaras, the air is only in the summer months more humid than in England.

III.—ON THE ADDITIONAL TABLES.

ATMOSPHERIC PRESSURE.

THE daily variation of pressure in India is extremely regular; the minima occur at about 4 A.M. and 5 P.M., the maxima at about 10 A.M. and 11 P.M., or, roughly speaking, at about one or two hours before sunrise, noon, sunset, and midnight; the morning maximum being greater than the evening one, and the evening minimum lower than the morning one. The difference between the mean

daily readings seldom exceeds ·2 inches, and the whole daily range is generally less than ·1 inch. The mean change of pressure from year to year is generally very small, and the change from month to month is very constant in different years, the maximum being in January and the minimum in June, the pressure increasing and decreasing regularly throughout the year, the difference being ·26 in the Presidencies of Madras and Bombay, and ·44 in that of Bengal generally. Locally, the distribution of pressure, from the account of Mr. Blanford, is as follows:—

Beginning with October, the month in which the south-west mansun terminates, the pressure is nearly uniform over Burmah, Bengal, Central, Northern, and Eastern India: in November, the pressure rises rapidly over the whole of this area, but more especially in two distinct areas, one being the elevated tract lying south of the Ganges, including Bandalkand, Chota Nagpur, and a part of Nagpur, up to Banaras on the north and down to Cuttack on the south; the others being an area in the Upper Panjab coinciding with the locus of lowest mean winter temperature. The intermediate Gangetic plains on the Gangetic delta, the Malwa plateau, and the flats of Southern Orissa, fall outside both of these areas. In December the general pressure is at its annual maximum, and in January it is nearly as high, all over India, but the pressure is less at Bombay and on the west coast than in Eastern India. It is probable that the fall of pressure with the approach of the hot weather is less rapid in the Panjab than in the Central Provinces and Bengal. In March, April, and May the maximum pressure is about Nagpur, and in the hill country about Hazaribagh it is lower than either on the delta and coast to the east and south-east, or in the Upper Provinces to the west and north-west. In June, the setting in of the south-west mansun is accompanied by a sudden fall of pressure; greater, however, in the Panjab than in the Nagpur region, so that the locus of minimum pressure is probably transferred to the former. In

June and July the pressure is nearly the same, and is the minimum of the year. During the continuance of the rains, the pressure rises but gradually, but in October the close of the mansun is marked by a more rapid rise, and in such a measure that the pressure is nearly equalized over the whole region.

A table of pressure is given among the additional meteorological tables, which are all taken from Mr. Blanford's Reports.

TEMPERATURE.

First, as regards the mean temperature of the air. The mean daily range of these temperatures at places at high elevations differs little from those at low elevations in the same latitude. The mean monthly temperatures at the same place do not, on the whole, undergo great variation.

The following are the most probable values of the effect on the mean monthly temperatures of an increase of one degree of latitude:—

October	$-0^\circ\cdot2$	April, no change.	May	$+0^\circ\cdot3$
November	$-0^\circ\cdot5$		June	$+0^\circ\cdot4$
December	$-1^\circ\cdot1$		July	$+0^\circ\cdot3$
January	$-1^\circ\cdot0$		August	$+0^\circ\cdot2$
February	$-0^\circ\cdot8$		September	$+0^\circ\cdot1$
March	$-0^\circ\cdot5$			

the greatest differences being in December; they exceed 30° in amount for the extremes of Madras and Bengal with regard to latitude. At moderate elevations, the mean temperature of the air at a place may be calculated from formulæ which allow for the effect of each degree of longitude, and apply to places within five degrees of latitude from each other; the effect of elevation above mean sea level, viz., 1° for 350 feet, may be also applied, and an average monthly mean temperature thus calculated for any place in India.

Secondly. As regards high temperature in the day. The decrease of high day temperature in India varies everywhere

regularly with the increase of elevation above mean sea level up to 9,000 feet, according to the following table :—

Height.	Decrease.	Height.	Decrease.
1000	$\frac{3}{4}^{\circ}$ to $4\frac{1}{2}^{\circ}$	6000	18° to $27\frac{3}{4}^{\circ}$
2000............	3° to $8\frac{3}{4}^{\circ}$	7000	21° to 36°
3000............	$6\frac{1}{2}^{\circ}$ to $13\frac{1}{4}^{\circ}$	8000	23° to 44°
4000............	$11\frac{1}{4}^{\circ}$ to 18°	9000	26° to 53°
5000............	15° to 23°		

the amounts given being maxima and minima in the year. There is also a regular monthly increase or decrease of high day temperature, due to an increase of one degree of latitude, thus :—

For November...	$-0^{\circ}\cdot 3$	For April.........	$+\ 0^{\circ}\cdot 2$
„ December ...	$-1^{\circ}\cdot 1$	„ May	$+\ 0^{\circ}\cdot 3$
„ January......	$-0^{\circ}\cdot 7$	„ June	$+\ 0^{\circ}\cdot 4$
„ February ...	$-0^{\circ}\cdot 7$	„ July	$+\ 0^{\circ}\cdot 4$
„ March	$-0^{\circ}\cdot 3$	„ August	$+\ 0^{\circ}\cdot 1$
		„ September ...	$+\ 0^{\circ}\cdot 2$
		„ October	$+\ 0^{\circ}\cdot 1$

The effect of longitude is inappreciable from June to August and for other months, westward stations have a higher day temperature than eastward by a difference of about half a degree for each degree of longitude.

Thirdly. As regards low temperature at night. The effect of latitude on low night temperature is almost inappreciable from May to September; but from November to March the effect is about one degree of temperature for each degree, and in April and October the effect is about half that; the northern stations being colder. The effect of an increase of one degree of east longitude is greatest in places having less than fifteen degrees of latitude; it amounts to a decrease of more than one degree and a half for each degree of greater east longitude in January and February, to a little less than that in March, to three-quarters of a degree in April, and to one quarter of a degree in May. After May a change takes place,

and from June to September those places with greater east longitude are from a quarter to half a degree warmer for each degree of longitude. The following table gives the decrease of night temperature due to increase of elevation up to 9,000 feet:

Height.	Decrease.	Height.	Decrease.
1000	$\frac{1}{2}°$ to $3\frac{1}{2}°$	6000............	13° to $22\frac{1}{4}°$
2000	$3\frac{1}{4}°$ to $6\frac{3}{4}°$	7000............	15° to 29°
3000	$5\frac{3}{4}°$ to $10\frac{1}{2}°$	8000............	17° to 35°
4000	$8\frac{1}{2}°$ to $14\frac{3}{4}°$	9000............	19° to 41°
5000	$10\frac{3}{4}°$ to 18°		

the amounts given being monthly maxima and minima in the year.

Some statistics of mean temperature of the air, of the temperature of solar radiation, black bulb thermometer in vacuo, and of grass radiation, for various places in India, will be found in the additional tables.

WIND AND SERENITY.

The phenomena of the mansuns and general winds of India being better studied from the charts of the large works on physical geography than from any brief account that the limits of this book would allow, it will be unnecessary here to enter into the subject. With regard to local observation of wind in India, comparatively little has been yet done. Mr. Chambers' "Winds of Bombay" gives some valuable information for the year 1867 in a novel form; and the two accompanying tables, taken from the report of Mr. Blanford for 1873, comprise everything else that is of much value. A table of serenity for a few places is also given.

In conclusion, the Meteorological Statistics of India are still too incomplete and irregular to lead to any very important scientific result—in fact, they do not yet arrive at the sufficiency required by the engineer; nevertheless, a judicious use of such data as we possess may, it is hoped, prevent the recurrence of such difficulties as have so frequently occurred from totally ignoring them.

TABLE

(Of Guyot, arranged by Blanford)

FOR FINDING THE RELATIVE HUMIDITY OF THE AIR,

FROM THE READINGS OF WET AND DRY BULB THERMOMETERS, SATURATION BEING 100.

FOR THE USE OF OBSERVERS.

Relative Humidity Table.

Wet Bulb Thermometer, t Fahrenheit	t—t', or difference of Wet and Dry Bulb Thermometers. t' below the Freezing Point, the Bulb covered with a Film of Ice.																						
	0·0	0·5	1·0	1·5	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8·0	8·5	9·0	9·5	10·0	10·5	11·0	11·5
	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.
0	100	85	70	57	43	31	19	7																
1	100	85	71	58	46	33	22	11																
2	100	86	73	60	48	36	25	14																
3	100	86	74	61	50	38	26	17																
4	100	87	75	63	52	41	30	20	11															
5	100	87	76	64	53	43	33	23	14															
6	100	88	76	66	55	45	35	26	17	9														
7	100	88	77	67	56	47	37	28	20	12														
8	100	89	78	68	58	48	39	31	22	15														
9	100	89	79	69	59	50	41	33	25	17	10													
10	100	89	79	70	61	52	43	35	28	20	13													
11	100	90	80	71	62	53	45	37	30	23	16													
12	100	90	81	72	63	56	47	40	32	25	19	12	6											
13	100	90	81	73	64	56	49	42	35	28	21	15	9											
14	100	91	82	73	65	58	50	43	37	30	24	18	12	7										
15	100	91	83	74	67	59	52	45	39	32	26	21	15	[illegible]	5									
16	100	91	83	75	68	61	54	47	41	35	29	23	18	13	8									
17	100	91	84	76	69	62	55	49	43	37	31	26	21	16	11	6								
18	100	92	84	77	70	63	57	50	44	39	33	28	23	18	14	9	5							
19	100	92	85	78	71	64	58	52	46	41	35	30	25	21	16	12	8	4						
20	100	92	85	78	72	65	59	54	48	43	37	32	28	23	19	15	11	7	3					
21	100	92	86	79	73	66	61	55	50	44	39	34	30	25	21	17	13	9	6	2				
22	100	93	86	80	73	68	62	56	51	46	41	36	32	28	23	19	16	12	8	5	2			
23	100	93	86	80	74	68	63	58	52	48	43	38	34	30	26	22	18	14	11	8	4			
24	100	93	87	81	75	69	64	59	54	49	44	40	36	32	28	24	20	17	13	10	7	4		
25	100	93	87	81	76	70	65	60	55	51	46	42	38	34	30	26	23	19	16	13	[illegible]	7	4	
26	100	94	88	82	76	71	66	61	56	52	48	43	39	35	32	28	25	21	18	15	12	9	6	[illegible]
27	100	94	88	82	77	72	67	62	58	53	49	45	41	37	34	30	27	23	20	17	14	11	9	6
28	100	94	88	83	78	73	68	63	59	55	51	47	43	39	35	32	29	25	22	19	16	14	11	8
29	100	94	89	83	78	74	69	64	60	56	52	48	44	41	37	34	31	27	24	21	19	16	13	11
30	100	94	89	84	79	74	70	65	61	57	53	49	46	42	39	35	32	29	26	23	21	18	15	13
31	100	94	89	84	80	75	71	66	62	58	55	51	47	44	40	37	34	31	28	25	23	20	17	15

Relative Humidity Table.

Wet Bulb Thermometer, t' Fahrenheit.	t—t', or difference of Wet and Dry Bulb Thermometers.																	
	0°.0	0°.5	1°.0	1°.5	2°.0	2°.5	3°.0	3°.5	4°.0	4°.5	5°.0	5°.5	6°.0	6°.5	7°.0	7°.5	8°.0	8°.5
	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity
32°	100	94	89	84	79	74	69	65	61	56	52	48	45	41	37	34	31	28
33	100	94	89	84	[illegible]	75	70	66	61	57	53	50	46	42	39	36	32	29
34	100	94	89	84	80	75	71	66	62	58	54	51	47	44	40	37	34	31
35	100	94	90	85	80	76	71	67	63	59	55	52	48	45	42	38	35	32
36	100	95	90	85	81	76	72	68	64	60	56	53	50	46	43	40	37	34
37	100	95	90	86	81	77	73	69	65	61	57	54	51	47	44	41	38	35
38	100	95	90	86	82	77	73	69	66	62	[illegible]	55	52	49	45	42	40	37
39	100	95	91	86	82	78	74	70	66	63	59	56	53	50	47	44	41	38
40	100	95	91	86	83	78	75	71	67	64	60	57	54	51	48	45	[illegible]	39
41	100	95	91	87	83	79	75	72	68	65	61	58	55	52	49	46	43	41
42	100	95	91	87	83	79	76	72	69	[illegible]	62	59	56	53	50	47	45	42
43	100	95	91	87	84	80	76	73	69	66	63	60	57	54	51	48	46	43
44	100	95	92	88	84	80	77	73	70	67	63	61	58	55	52	49	47	44
45	100	96	92	88	84	81	77	74	71	67	64	61	59	56	53	50	48	45
46	100	96	92	88	85	81	78	74	71	68	65	62	59	57	54	51	49	46
47	100	96	92	88	85	81	78	75	72	68	66	63	60	57	55	52	50	47
48	100	96	92	89	85	82	79	75	72	69	66	64	61	58	56	53	51	48
49	100	96	92	89	86	82	79	76	73	70	67	64	62	59	56	54	52	49
50	100	96	92	89	86	82	79	76	73	70	68	65	62	60	57	55	52	50
51	100	96	93	89	86	83	80	77	74	71	68	66	63	60	58	56	53	51
52	100	96	93	89	86	83	80	77	74	71	69	66	64	61	59	56	54	52
53	100	96	93	90	86	83	80	78	75	72	69	67	64	62	59	57	55	53
54	100	96	93	90	87	84	81	78	75	72	70	67	65	62	60	58	56	53
55	100	96	93	90	87	84	81	78	76	73	70	68	65	63	61	59	56	54
56	100	96	93	90	87	84	81	79	76	73	71	68	66	64	61	59	57	55
57	100	96	93	90	87	85	82	79	76	74	71	69	67	64	62	60	58	56
58	100	96	93	90	88	85	82	79	77	74	72	69	67	65	63	60	58	56
59	100	96	94	91	88	85	82	80	77	75	72	70	68	65	63	61	59	57
60	100	97	94	91	88	85	83	80	78	75	73	70	68	66	64	62	60	58
61	100	97	94	91	88	85	83	80	78	75	73	71	69	66	64	62	60	58
62	100	97	94	91	88	86	83	81	78	76	74	71	69	67	65	63	61	59
63	100	97	94	91	89	86	83	81	79	76	74	72	70	67	65	63	61	59
64	100	97	94	91	89	86	84	81	79	77	74	72	70	68	66	64	62	60
65	100	97	94	91	89	86	84	81	79	77	75	72	70	68	66	64	62	61
66	100	97	94	92	89	87	84	82	79	77	75	73	71	69	67	65	63	61
67	100	97	.	92	89	87	84	82	[illegible]	78	75	73	71	69	67	65	63	62

Relative Humidity Table.

Wet Bulb Thermometer, t' Fahrenheit.	t—t', or difference of Wet and Dry Bulb Thermometers.																	
	0·0	0·5	1·0	1·5	2·0	2·5	3·0	3·5	4·0	4·5	5·0	5·5	6·0	6·5	7·0	7·5	8·0	8·5
	Humidity	Humidity	Humidity.	Humidity	Humidity.	Humidity.	Humidity.	Humidity	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity	Humidity.	Humidity
68	100	97	94	92	89	87	85	82	[illegible]	78	76	74	72	70	68	66	64	62
69	100	97	94	92	89	87	85	82	80	78	76	74	72	70	68	66	64	63
70	100	97	94	92	90	87	85	83	81	78	76	74	72	70	68	67	65	63
71	100	97	95	92	90	87	85	83	81	79	77	75	73	71	69	67	65	64
72	100	97	95	92	[illegible]	88	85	83	81	79	77	75	73	71	69	67	66	64
73	100	97	95	92	90	88	86	83	81	79	77	75	73	71	70	68	66	64
74	100	97	95	92	90	88	86	84	81	79	77	76	74	72	70	68	66	65
75	100	97	95	92	90	88	86	84	82	80	78	76	74	72	70	69	67	65
76	100	97	95	93	90	88	86	84	82	80	78	76	74	72	71	69	67	66
77	100	97	95	93	[illegible]	88	86	84	82	80	78	76	75	73	71	69	68	66
78	100	97	95	93	91	88	86	84	82	80	78	77	75	73	71	70	68	66
79	100	97	95	93	91	89	87	85	83	81	79	77	75	73	72	70	68	67
80	100	97	95	[illegible]	91	89	87	85	83	81	79	77	75	74	72	70	69	67
81	100	97	95	93	91	89	87	85	83	81	79	77	76	74	72	71	69	67
82	100	97	95	93	91	89	87	85	83	81	79	78	76	74	73	71	69	68
83	100	97	95	93	91	89	87	85	83	[illegible]	80	78	76	74	73	71	70	68
84	100	97	95	93	91	89	87	85	83	82	80	78	76	75	73	71	70	68
85	100	97	95	93	91	89	87	85	84	82	80	78	77	75	73	72	70	69
86	100	97	95	93	91	89	87	86	84	82	80	79	77	75	74	72	70	69
87	100	97	95	93	91	90	88	86	84	82	80	79	77	76	74	72	71	69
88	100	97	95	93	92	90	88	86	84	82	81	79	77	76	74	73	71	70
89	100	97	95	94	92	90	88	86	84	83	81	79	78	76	74	73	71	70
90	100	98	96	94	92	90	88	86	84	83	81	79	78	76	75	73	72	70
91	100	98	96	94	92	90	88	86	85	83	81	80	78	76	75	73	72	70
92	100	98	96	94	92	90	88	86	85	83	81	80	78	77	75	74	72	71
93	100	98	96	[illegible]	92	90	88	87	85	83	82	80	78	77	75	74	72	71
94	100	98	96	94	92	90	88	87	85	83	82	80	79	77	75	74	73	71
95	100	98	96	[illegible]	92	90	88	87	85	83	82	80	79	77	76	74	73	71
96	100	98	96	94	92	90	89	87	85	84	82	80	79	77	76	74	73	72
97	100	98	96	94	92	90	89	87	85	84	82	81	79	78	76	75	73	72
98	100	98	96	94	92	90	89	87	85	84	82	81	79	78	76	75	73	72
99	100	98	96	94	92	91	89	87	86	84	82	81	79	78	76	75	74	72
100	100	98	96	94	92	91	89	87	86	84	83	81	80	78	77	75	74	72
101	100	98	96	94	92	[illegible]	89	87	86	84	83	81	80	78	77	75	74	73
102	100	98	96	94	92	91	89	88	86	84	83	81	80	78	77	76	74	73
103	100	98	96	94	93	91	89	88	86	84	83	81	80	79	77	76	74	73
104	100	98	96	94	93	91	89	88	86	85	83	82	80	79	77	76	75	73

Relative Humidity Table.

Wet Bulb Thermometer, t′ Fahrenheit	t − t′, or difference of Wet and Dry Bulb Thermometers.																	
	9°·0	9°·5	10°·0	10°·5	11°·0	11°·5	12°·0	12°·5	13°·0	13°·5	14°·0	14°·5	15°·0	15°·5	16°·0	16°·5	17°·0	17°·5
	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity	Humidity.	Humidity	Humidity.	Humidity
32°	25	22	19	16	13	11	8	6	4									
33	26	23	21	18	15	13	10	8	6	4								
34	28	25	22	20	17	15	12	10	8	6	4							
35	30	27	24	21	19	16	14	12	10	8	6	4						
36	31	28	26	23	21	18	16	14	12	10	8	6	4					
37	33	30	27	25	22	20	18	16	13	11	10	8	6	4				
38	34	31	29	26	24	22	19	17	15	13	11	10	8	6	4	3		
39	35	33	30	28	26	23	21	19	17	16	13	11	10	8	6	5	3	2
40	37	34	32	29	27	25	23	21	19	17	15	13	11	10	8	6	5	3
41	38	36	33	31	29	27	24	22	20	18	17	15	13	11	10	8	7	5
42	39	37	35	32	30	28	26	24	22	20	18	16	15	13	12	10	8	7
43	41	38	36	34	32	29	27	25	23	22	20	18	16	15	13	11	10	9
44	42	39	37	35	33	31	29	27	25	23	21	19	18	16	15	13	12	10
45	43	41	38	36	34	32	30	28	26	24	22	21	19	18	16	15	13	12
46	44	42	39	37	35	33	31	30	28	26	24	22	21	19	17	16	15	13
47	45	43	41	39	37	35	33	31	29	27	25	24	22	20	19	17	16	15
48	46	44	42	40	38	36	34	32	30	28	27	25	23	22	20	19	17	16
49	47	45	43	41	39	37	35	33	31	30	28	26	25	23	22	20	19	17
50	48	46	44	42	40	38	36	34	32	31	29	27	26	24	23	22	20	19
51	49	47	45	43	41	39	37	35	34	32	30	29	27	26	24	23	21	20
52	50	48	46	44	42	40	38	36	35	33	31	30	28	27	25	24	23	21
53	51	49	47	45	43	41	39	37	36	34	32	31	29	28	27	25	24	23
54	51	49	47	46	44	42	40	38	37	35	34	32	30	29	28	26	25	24
55	52	50	48	46	45	43	41	39	38	36	35	33	32	30	29	27	26	25
56	53	51	49	47	45	44	42	40	39	37	35	34	33	31	30	28	27	[illegible]
57	54	52	50	48	46	44	43	41	40	38	36	35	34	32	31	29	28	27
58	55	52	51	49	47	45	44	42	40	39	37	36	34	33	32	30	29	28
59	55	53	53	51	50	48	46	44	43	41	40	38	37	35	34	33	31	30
60	56	54	52	50	48	47	45	44	42	41	39	38	36	35	34	32	31	30
61	57	55	53	51	49	48	46	44	43	41	40	38	37	36	34	33	32	31
62	57	55	53	52	50	48	47	45	44	42	41	39	38	37	35	34	33	32
63	58	56	54	52	51	49	47	46	44	43	41	40	39	37	36	35	34	32
64	58	56	54	53	51	50	48	47	45	44	42	41	40	39	37	36	34	33
65	59	57	55	54	52	50	49	47	46	44	43	42	40	39	38	36	35	34
66	59	58	56	54	53	51	49	48	47	45	44	42	41	40	38	37	36	35
67	60	58	56	55	53	52	50	49	47	46	44	43	42	40	39	38	37	36

Relative Humidity Table.

Wet Bulb Thermometer, t′ Fahrenheit.	t—t′, or difference of Wet and Dry Bulb Thermometers.—Fahrenheit.																	
	9°·0	9°·5	10°·0	10°·5	11°·0	11°·5	12°·0	12°·5	13°·0	13°·5	14°·0	14°·5	15°·0	15°·5	16°·0	16°·5	17°·0	17°·5
	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.
68°	60	59	57	55	54	52	51	49	48	46	45	44	42	41	40	39	37	36
69	61	59	58	56	54	53	51	50	48	47	46	[illegible]	43	42	41	39	38	37
70	61	60	58	56	55	53	52	50	49	48	46	45	44	42	41	40	39	38
71	62	60	59	57	55	54	52	51	50	48	47	46	44	43	42	41	40	38
72	62	61	59	58	56	54	53	52	50	49	48	46	45	44	43	41	40	39
73	63	61	60	58	56	55	54	52	51	[illegible]	48	47	46	44	43	[illegible]	41	40
74	63	62	60	58	57	56	54	53	51	50	49	47	46	45	44	43	41	40
75	64	62	60	59	57	56	55	53	52	51	49	48	47	46	44	43	42	41
76	64	62	61	59	58	56	55	54	[illegible]	51	50	48	47	[illegible]	45	44	[illegible]	42
77	64	63	61	60	58	57	56	54	53	52	50	49	48	47	45	[illegible]	43	42
78	65	63	62	60	59	57	56	55	53	52	51	50	48	47	46	45	44	43
79	65	64	62	61	59	58	56	55	54	53	51	50	[illegible]	48	47	46	44	43
80	66	64	63	61	60	58	57	56	54	53	52	51	49	48	47	46	45	44
81	66	64	63	61	60	59	57	56	55	53	52	51	[illegible]	49	48	48	45	44
82	66	65	63	62	60	59	58	56	55	54	53	51	50	49	48	47	46	45
83	67	65	64	62	61	59	58	57	56	54	53	[illegible]	51	50	48	47	46	45
84	67	65	64	63	61	60	59	57	56	55	[illegible]	52	51	50	[illegible]	48	47	46
85	67	66	64	63	62	60	59	58	56	[illegible]	54	53	52	[illegible]	49	48	47	46
86	68	66	65	63	62	61	59	58	57	56	54	53	52	51	50	49	48	47
87	68	66	65	64	62	61	[illegible]	58	57	56	55	54	52	51	50	49	48	47
88	68	67	65	64	63	61	60	59	58	56	55	54	53	52	51	50	49	48
89	68	67	66	64	63	62	60	59	58	57	56	54	53	52	51	50	49	48
90	69	67	66	65	63	62	61	59	58	57	56	55	54	53	51	50	49	48
91	69	68	66	65	64	62	61	60	59	57	56	55	54	53	52	51	50	49
92	69	68	67	65	64	63	61	60	59	58	57	55	54	53	52	51	50	49
93	69	68	67	65	64	63	62	60	[illegible]	58	57	56	[illegible]	54	53	52	51	50
94	70	68	67	66	64	63	63	61	60	58	57	56	55	54	53	52	51	50
95	70	69	67	66	65	64	62	61	[illegible]	59	58	57	55	54	53	52	51	50
96	70	69	68	66	65	64	63	61	60	59	58	57	56	55	54	53	52	51
97	70	69	68	67	65	64	63	62	61	59	58	57	56	55	[illegible]	53	52	51
98	71	69	68	67	66	64	63	62	61	60	59	57	56	55	54	53	52	51
99	71	70	68	67	[illegible]	65	[illegible]	62	61	60	59	58	57	56	55	54	53	52
100	71	70	69	67	66	65	64	63	62	60	59	58	57	56	55	54	53	52
101	71	70	69	66	66	65	64	63	62	61	59	[illegible]	57	56	55	54	53	52
102	72	70	69	68	67	66	64	63	62	61	60	59	58	57	56	55	54	53
103	72	71	69	68	67	66	64	63	62	61	60	59	58	57	56	55	54	53
104	72	71	69	68	67	66	65	64	63	61	60	59	58	57	56	55	54	53

Relative Humidity Table.

Wet Bulb Thermometer, t′ Fahrenheit.	t—t′, or difference of Wet and Dry Bulb Thermometers.—Fahrenheit.																	
	18·0	18·5	19·0	19·5	20·0	20·5	21·0	21·5	22·0	22·5	23·0	23·5	24·0	24·5	25·0	25·5	26·0	26·5
	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.	Humidity.
32																		
33																		
34																		
35																		
36																		
37																		
38																		
39																		
40	2																	
41	4	3	1															
42	6	4	3	2														
43	7	6	5	3	2													
44	9	7	6	5	4	3	2											
45	10	9	8	7	5	4	3	2	1									
46	12	11	9	8	7	6	5	4	3	2								
47	13	12	11	10	9	7	6	5	4	3	2	1						
48	15	13	12	11	10	9	8	7	6	5	4	3	2	1				
49	16	15	14	12	11	10	9	8	7	6	5	4	3	3	2	1		
50	18	16	15	14	13	12	11	10	9	8	7	6	5	4	3	2	2	1
51	19	18	16	15	14	13	12	11	10	9	8	7	6	5	4	4	3	2
52	20	19	18	16	15	14	13	12	11	10	9	8	7	7	6	5	4	3
53	21	20	19	18	16	15	14	13	12	11	10	9	9	8	7	6	5	5
54	22	21	20	19	18	17	16	14	14	13	12	11	10	9	8	7	7	6
55	24	22	21	20	19	18	17	16	15	14	13	12	11	10	9	9	8	7
56	25	23	22	21	20	19	18	17	16	15	14	13	12	11	11	10	9	8
57	26	24	23	22	21	20	19	18	17	16	15	14	13	13	12	11	10	9
58	27	25	24	23	22	21	20	19	18	17	16	15	14	14	13	12	11	10
59	28	26	25	24	23	22	21	20	19	18	17	16	16	15	14	13	12	12
60	29	27	26	25	24	23	22	21	20	19	18	17	17	16	15	14	13	13
61	30	28	27	26	25	24	23	22	21	20	19	18	17	17	16	15	14	14
62	30	29	28	27	26	25	24	23	22	21	20	19	18	18	17	16	15	15
63	31	30	29	28	27	26	25	24	23	22	21	20	19	19	18	17	16	15
64	32	31	30	29	28	27	26	25	24	23	22	21	20	19	19	18	17	16
65	33	32	31	30	29	28	27	26	25	24	23	22	21	20	20	19	18	17
66	34	33	32	30	29	28	27	27	26	25	24	23	22	21	20	20	19	18
67	34	33	32	31	30	29	28	27	26	25	25	24	23	22	21	20	20	19

Relative Humidity Table.

Wet Bulb Thermometer, t′ Fahrenheit.	t—t′, or difference of Wet and Dry Bulb Thermometers.—Fahrenheit.																	
	18°·0	18°·5	19°·0	19°·5	20°·0	20°·5	21°·0	21°·5	22°·0	22°·5	23°·0	23°·5	24°·0	24°·5	25°·0	25°·5	26°·0	26°·5
	Humidity.	Humidity	Humidity.	Humidity	Humidity	Humidity.	Humidity.	Humidity	Humidity	Humidity	Humidity.	Humidity.	Humidity.	Humidity.	Humidity	Humidity.	Humidity.	Humidity
68	35	34	33	32	31	30	29	28	27	26	25	25	24	23	22	21	21	[illegible]
69	36	35	34	33	32	31	30	29	28	27	26	[illegible]	24	[illegible]	23	22	21	21
70	37	36	35	33	32	31	31	30	29	28	27	26	25	24	24	23	22	21
71	37	36	35	34	33	32	31	30	29	29	28	27	26	25	24	24	23	22
72	38	37	36	35	34	33	32	[illegible]	30	29	28	28	27	26	25	24	24	23
73	39	38	37	36	35	34	33	32	31	30	29	28	27	27	26	25	24	24
74	39	38	37	36	35	34	33	32	31	31	30	29	28	27	27	26	25	24
75	40	39	[illegible]	37	36	35	34	33	32	31	[illegible]	30	29	28	27	26	26	25
76	40	39	[illegible]	37	36	35	35	34	33	[illegible]	31	30	29	29	28	27	26	26
77	41	40	39	38	37	36	[illegible]	34	33	33	32	31	30	29	28	28	27	26
78	42	41	40	39	38	37	36	35	34	33	32	31	31	30	29	28	[illegible]	27
79	42	41	40	39	38	37	36	[illegible]	35	34	33	32	31	30	30	29	28	27
80	43	42	41	40	39	38	37	36	35	34	33	33	32	[illegible]	30	29	29	28
81	43	43	41	40	39	38	37	37	36	35	34	33	32	32	31	30	29	29
82	44	43	42	41	40	39	38	37	36	[illegible]	35	34	33	32	31	31	30	[illegible]
83	44	43	42	41	40	39	39	38	37	[illegible]	35	[illegible]	33	33	32	[illegible]	30	30
84	45	44	43	42	41	40	39	38	37	36	36	35	34	[illegible]	32	32	31	30
85	45	[illegible]	43	42	41	[illegible]	40	39	38	37	36	35	35	34	33	32	32	31
86	46	45	44	43	42	41	40	39	38	37	37	36	35	34	33	33	32	31
87	46	[illegible]	44	43	42	41	40	[illegible]	39	38	37	36	[illegible]	35	34	33	33	32
88	47	46	45	44	43	42	41	[illegible]	[illegible]	38	38	37	36	35	34	34	33	32
89	47	46	45	44	43	42	41	41	40	39	38	37	36	36	35	34	33	[illegible]
90	47	46	46	45	44	43	42	41	40	39	38	38	37	36	35	35	34	33
91	48	47	46	45	44	43	42	41	41	40	39	38	37	37	36	[illegible]	34	34
92	48	47	[illegible]	45	44	44	43	42	41	40	39	39	[illegible]	37	36	36	35	34
93	49	48	47	46	45	44	43	[illegible]	41	41	[illegible]	39	38	37	37	36	35	35
94	[illegible]	48	47	46	46	44	43	43	42	41	[illegible]	39	39	38	37	36	36	[illegible]
95	49	48	47	47	46	45	44	43	42	41	41	40	39	38	37	37	[illegible]	35
96	50	49	48	47	46	45	44	43	43	42	41	40	39	39	38	37	37	36
97	50	49	48	47	46	46	45	44	43	42	41	41	40	39	38	38	37	36
98	50	49	49	48	47	46	[illegible]	44	43	43	42	41	[illegible]	39	[illegible]	38	37	37
99	51	50	49	48	47	46	45	45	[illegible]	43	[illegible]	41	41	40	38	38	38	37
100	51	50	49	48	47	47	[illegible]	45	44	43	43	42	41	40	39	39	[illegible]	37
101	51	50	50	[illegible]	[illegible]	47	46	45	44	44	[illegible]	[illegible]	41	41	40	39	38	[illegible]
102	52	51	50	[illegible]	48	47	46	46	45	44	43	42	42	41	40	40	39	38
103	52	51	50	49	48	48	47	46	45	44	44	[illegible]	42	41	41	40	39	38
104	52	51	51	50	49	48	47	46	45	45	44	43	42	42	41	40	40	39

TABLE OF CONTENTS.

PART I.

MANUAL.

WORKING TABLES.

MISCELLANEOUS TABLES AND DATA.

PART II.

HYDRAULIC STATISTICS.

INDIAN METEOROLOGICAL STATISTICS.